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## Slope Stability Enhancement of an Upstream Tailings Dam: Laboratory Testing and Numerical Modelling

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Graduate Program in Civil and Environmental Engineering  
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## **Abstract**

Mine tailings are the byproduct of mining activities, which need to be disposed of once the ore is extracted. They can be disposed of in either dry or wet forms. The latter is most common with the tailings being disposed of in the form of slurry inside retention structures. The retention structure may be a natural, manmade, or built dam, which is the case in most current mining locations. In this thesis, improving the stability of an upstream tailings dam using soil additives is investigated. The experimental phase of this study involved laboratory tests conducted to characterize mine tailings and to investigate any change in their properties upon stabilization with non-traditional and traditional additives; namely, emulsified polymer and a mixture composed of Cement Kiln Dust, CKD, and re-cycled Gypsum. Afterwards, the soil modified parameters are used to establish a finite element model employing the commercial code PLAXIS 2D to simulate the behavior of the improved soil when a tailings dam is formed. The numerical model demonstrated that utilizing a CKD: B mix increased the overall stability of the tailings impoundment and indicated it is very useful to construct robust dams, yet is still environmentally friendly.

## **Keywords;**

Slope stability, upstream tailings dams, Plaxis 2D, emulsified polymers, CKD, recycled gypsum, numerical modelling.

## **Co-Authorship Statement**

This thesis has been prepared in accordance with the regulation of paper based format stipulated by the school of Graduate and Postdoctoral Studies at the University of Western Ontario. All modeling process, physical testing, data analysis, and writing of initial version of this thesis were carried out by the candidate himself under supervision of his research advisor, Professor Hesham El Naggar. Versions of Chapters 3 and 4 will be submitted for possible publication in peer-reviewed journals.

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## Glossary

$c'$	Effective cohesion of soil for Mohr-Coulomb failure criterion
$\Phi'$	Angle of internal friction angle
$C_v$	Coefficient of consolidation
$D_{50}$	Mean beach particle size
$D_{10}$	Effective particle size
$E$	Young's Modulus
$e$	Void ratio
$e_o$	Void ratio at zero effective stresses
$C_c$	Compression index
$C_s$	Swelling index
$m_v$	Coefficient of compressibility
$\tau$	Shear strength of the material using Mohr-Coulomb failure criterion
$G_s$	Specific gravity
$S_u$	Undrained peak shear strength
UCS	Unconfined compressive strength
$\sigma'$	Effective stress
$D_r$	Relative density
$u$	Hydrostatic pore water pressure
$k$	Hydraulic conductivity
EPP	Excess pore water pressure
S.F	Safety factor
$t$	time
ICP	Inductive coupled plasma
XRF	X-Ray Fluorescence
B	Recycled gypsum (basanite)

CKD	Cement kiln dust
P	Commercial gypsum (plaster)
DS	Direct shear (test)
$\gamma$	Material's unit weight
Wc%	Water content
LL	Liquid limit
PL	Plastic limit
LI	Liquidity index
Cc	Coefficient of uniformity
Cu	Coefficient of curvature
USCS	Unified soil classification system
TSF	Tailings storage facility

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# **Chapter1**

## **“Introduction and Research Objectives”**

### 1.1 Introduction

#### 1.1.1 Preface

Tailings are byproducts or “wastes” of mining activities. They result from the crushing and grinding of rocks done in order to extract the “ore”. Typically, the ore percentages within the raw material are between 30 to 0.4%, which means massive amounts of mined materials are disposed of daily (Zardari, 2013). A single mine may produce tailings in the order of 1 to 6 million cubic meters per year. The range of increase in height of tailings deposits depends on the area available and the amount of yielding. The reported heightening rate is in the order of 2-9 m yearly. This situation has resulted in the construction of super giant tailings storage facilities, up to 250m high has been reported (Villavicencio, 2013). As tailings are used in the construction of these containment facilities, the knowledge of their properties becomes crucial for the safe design of the impoundments they form.

All tailings containment dam types are built in stages. Therefore, it is necessary to carefully evaluate the loading/excess pore pressure (EPP) generation, for wet disposal, and the subsequent consolidation behavior. Because of the problem complexity and the large scale of tailings dams, numerical modeling is usually pursued to aid in understanding the expected effects of this loading cycle on the tailings dam stability when staged construction is followed (e.g. Zardari, 2013; Orman et al., 2013).

The upstream construction method is the dominant construction method employed in practice because it is the most economic and efficient way for disposal of large amounts of tailings (Priscu, 1999; Ates, 2013; Naeini et al., 2011, Rout et al., 2013). However, this method was abandoned in some countries (e.g. Chile) due to its susceptibility to failure should an earthquake

take place (Villavicencio, 2013). Thus, some mine companies are forced to adopt other methods of construction, which makes the construction of containments much more expensive and time consuming. Nevertheless, mine operators are inclined to adopt the upstream construction technique due to its cost effectiveness and simplicity. Therefore, several soil improvement methods are explored to help enhance the safety of tailings dams constructed using the upstream construction technique.

The effect of soil treatment on the stability of tailings dams is assessed in the current study in terms of the overall safety factor. The main objective of this thesis is to assist mine operators, who mostly choose upstream tailings method due to its simplicity and economy, to meet the safety requirements of the code enforced, and to protect the people and environment surrounding mine activities.

## **1.2 Background**

### **1.2.1 Main Tailings Dams**

Significant research done on conventional earth dam performance enabled a deep scrutiny of the adopted practice for their design and construction, consequently, amendments were done where necessary. Therefore, sound design, management guidelines, and construction practices were thoroughly revised and have been established. Unlike conventional earth dams, tailings dams have received little attention by researchers until recently. On the other hand, the literature contains plenty of reported tailings dams' failures (Rico et al., 2008; Davis, 2002). Therefore, thorough analysis and assessment are necessary to find out the prevailing modes of failures and their causes.



Generally, tailings dams are composed of fine or granular soils usually characterized by low shear strength and varying particle size distribution. The properties of tailings are highly dependent on their parent materials, the processes they undergo until the ore is extracted, and the disposal method (Moghaddam, 2004; James et al., 2002; Priscu, 1999). They are mostly fine materials with angular texture with particle size range between fine sand to clay. They typically have low hydraulic conductivity due to their angular particles shape, which affects their drainage and their response to any hydraulic gradient change. Likewise, their shear strength parameters are affected by the crushing and grinding and by the properties of the parent rock. Additionally, the shear strength of the tailings deposit is influenced by how tightly packed the particles, which will depend on their sedimentation process, particles roundness, velocity of sedimentation and particles fineness.

The large quantities of tailings make it burdensome task for mine operators to tackle on less laborious and cost effective way. Therefore, tailings are mostly mixed with water and pumped to a pond that is contained by artificial or natural impoundments, or a combination of the two. There are numerous methods for the disposal of tailings; nonetheless, the predominant one is the formation of tailings dams. Different types of tailings dams are employed depending on the amount of sand yield. However, following no or little geotechnical guidelines in the initial practice of their construction made them prone to failure events that endanger the environment and, if there is any, the downstream inhabitants (Jeyapalan, 1980). The hazard level associated with tailings dam failure will depend on many factors, including, but not limited to: the volume released and its travel distance; the level of toxicity of the released volume; the closeness of any river or stream; and the existence of close human habitats. For instance, the failure events of El Cobre dam, Chile (1965), an iron tailing dam in Shanxi province, China (2008), and Buffalo

Creek dam, West Virginia, USA (1967) are three examples of those calamities which have been reported to claim more than 550 people's lives (Jeyapalan, 1980; Yin et al, 2011; Villavicencio, 2013).

Tailings dams operations can be classified into active, inactive, and inactive but monitored.

Many incidents and/or failures of tailings dams reported in the literature resulted mainly from the following: slope instability, liquefaction, increase in the phreatic level, piping, and overtopping (Zardari, 2013; Rico, 2008). Therefore, the scope of this thesis is to first gather and analyze information about tailings and their disposal, review the adopted practice and the associated problems with it and bring solutions accordingly.

### 1.2.2 Failure Triggering Mechanisms:

Villavicencio et al. (2013) studied the incidents and failures that took place over a 100 years period and reported that both active and abandoned tailings dams experienced failures. They reported that tailings dams can fail due to one of the following causes: the construction method (upstream method accounts for the majority of sand tailings dams' failure); poor compaction, high fine contents in the cyclone sand, and the elevated pore water pressure (Rico et al., 2008; Davis, 2002). Those causes can initiate the dam instability and consequently lead to incidents and/or failure. Incidents or failure may happen when excessive deformation occurs at the crest or toe of the embankment as a result of construction on weak foundation material such as soft clay. Additionally, failure may happen when the construction sequence does not allow the excess pore water pressure to dissipate, leading to elevated water table in the dam body, especially if not considered or predicted in the design stage. Elevated water table can also happen after a rain storm or high surface run-off. Failure can also happen due to earthquake loading, which causes cyclic shearing forces that could liquefy all or part of the dam body or its foundation soil and,

consequently, reduces the available shear strength. For example, Villavicencio et al. (2013) reported that most of Chilean sand tailings dams were observed to have clear water pond close to the crest of the retaining dyke. This is an indication of liquefied material and explicit outcome of raised water table and poor liquefaction countermeasure (Villavicencio et al., 2013).

Furthermore, slope instability during earthquakes may occur due to seismic inertia forces, especially when the slope has a low static factor of safety approaching a unity. Also, static liquefaction can take place when the time laps between two consecutive layer placements is not adequate to allow for pore water pressure generated to dissipate resulting in full or partial liquefaction for the dam body and/or the foundation due to the negation of effective stresses. It is believed that this phenomenon accounts for the majority of reported incidents of failure as those tailings dams are normally consolidated and their stability depends on the effective shear strength provided at the slip surface; when the effective strength decreases, the factor of safety decreases. It is surprising that most of the incidents and/or failures that happened in Chile are for dams having crest height less than 40 meters. Conversely, the Mauro dam, located in Central part of the seismic active region in Chile, sustained the earthquake of Feb.2010 having a magnitude  $M_w = 8.5$  because of its good drainage system along with good design and construction practice, even though it was 150m high at the earthquake time (Villavicencio et al., 2013). This clearly indicates the importance of good design and construction practice. Therefore, it is of utmost importance for the mining industry to design tailings dams to meet safety requirements implementing construction methodologies that ensure satisfactory performance under different loading conditions.

### 1.2.3 Regulations

Recent incidents and failures have increased public and practitioners awareness of the risks associated with the mining industry and thus urged the regulatory bodies to enforce sound regulations and rules for mine tailings disposal. Failures of tailings dams of El Cobre, Chile (1965), an iron tailings dam in China (2008), and Buffalo Creek, West Virginia, USA (1967) are three examples of those calamities which have been reported to claim more than 550 people's lives (Villavicencio, 2013; Yin et al, 2011; Jeyapalan, 1980). A review of the failure events spanning throughout the last century gave an insight of the importance of understanding mine tailings dams' behavior upon loading, wetting-drying cycles and at the onset of an earthquake event. Those studies were then used to establish guidelines for the design and maintenance that mine operators need to comply with. As the construction sequence covers the entire life of the mine operation, which lasts tens of years, the dam stability is governed by several factors: heavy rainfall, overtopping, flooding, seismic event and liquefaction. Researchers and mining companies have consequently been involved in gathering and assessing information about failure incidents. Throughout the past 3 decades, lessons learnt from previous incidents were the major driving factors behind reforming mine by-products disposal's regulations. For instance, in Chile, the regulations of mining byproducts disposal have been modified after an earthquake struck the west part of the region in 1986 and initiated slope instabilities at some tailings dams. Major contributions on tailings storage facility guidelines is credited to: Canada, Australia, and South Africa, as they have more than 1000 tailings dams (Engels, 2012). Worldwide, it is estimated that tailings dams are in the order of tens of thousands (Zardari, 2011, Yin et al, 2011).

There are two associations that concern tailings dams' guidelines in Canada, which have common goal towards tailings dams' safety, operation and closure. Namely, they are the Mining

Association of Canada (MAC; 1998 and 2003) and Canadian Dam Association (CDA, 1999). Nonetheless, the tailings dams' regulation in Canada is controlled by provinces within which they are located and additional restriction and/or guidelines may apply except for the Uranium tailings facilities which are controlled by the federal government. Similarly, in Australia, each state has its own guideline to control tailings disposal. The International Commission on Large Dams (ICLUD) has established a committee concerning tailings dams and has published eleven bulletins on their management, operation and closure. All these documents are collaborative efforts and were established by gathering information from published work and research.

### 1.3 Objectives and Scope of Research

The main objective of this research is to establish an efficient soil treatment scheme that can help enhance the stability of tailings dams constructed using the upstream method. In order to achieve this objective, it is necessary to investigate the beneficial effects of different soil additives, both non-traditional and traditional, with regard to enhancing the stability of tailings dams. Therefore, the study has two main components: an experimental component and a numerical investigation. The experimental component involves the study of employing different additives to improve the shear strength of tailings material in a laboratory setup. The numerical investigation is focused on evaluating the stability of tailings dams considering both untreated tailings and treated tailings. The efficiency of the tailings treatment is then evaluated in terms of the increase in the factor of safety of the tailings dam. A baseline had to be established in order to compare the stability of the improved tailings with respect to the stability of untreated tailings. Typically, tailings dams are constructed to retain the “wet tailings” that resulted from the mining activities. This kind of facility is conventionally referred to as a tailings storage facility (TSF). A typical TSF would encompass the tailings dam plus the contained tailings. Once the discovered

mineral(s) is/are feasible to extract and before the mining activities commence, a starter dike is usually built from borrowed materials to establish a pond for the first years of mining. As the pond starts to fill up, the tailings dam has to move upward in one of three directions, which defines the name of construction method followed. If the tailings dam center is kept and the crest is raised over the initial dike, then centerline method is adopted. If the center is changed to the upstream or to the downstream, then upstream or downstream method, respectively, of construction has been followed. The focus of this thesis is the upstream tailings dams, in which case the contained tailings are often disposed of in a form of slurry by spigot(s) to ease the dumping process and to reduce the cost of transportation. Therefore, water contents are usually very high especially near the surface for newly deposited materials yielding a very weak material. Over time, some of those contained tailings sediments become part of the tailing dam (Bjelkevick, 2005). As the mining activities continue, so does the construction of the TSF. The spigot(s) is progressively moved towards the TSF center and upward with time for upstream tailings dams; thus, the encapsulated tailings materials within the critical slip surface get larger and larger. Therefore, the properties of the tailings become critical from a stability point of view. In the current study, the evaluated tailings properties in addition to the unit weight are: friction angle, cohesion, Young's modulus and hydraulic conductivity. In an upstream built dam, the improvement sought after should manifest itself in the form of increased cohesion of the tailings matrix, otherwise a non-cohesive material due to the nature of deposition being in a slurry form that results in a normally consolidated deposit. This concept is commonly referred to as consolidated tailings or composite tailings, CT (Beier, 2015).

### 1.4 Research Methodology

To accomplish the objectives of the research, the following methodology is followed;

- 1- Conduct a comprehensive literature review of tailings dams, their method of construction and properties.
- 2- Assess the statistical data published in the literature related to the types of TSF and the merits and demerits of each type, and define the TSF type that has constructability advantage while having the potential to significantly benefit from tailings treatment.
- 3- Upstream tailings dam was selected based on the research efforts stated above.
- 4- Given that the literature review indicated that excess pore water pressure is the leading cause of instability incidents, effective stress analysis (ESA) of tailings dams is adopted in the study.
- 5- An experimental program involving testing treated and untreated specimens of tailings material retrieved from mines in Northern Ontario. The tailings treatment investigated included utilizing both traditional and non-traditional additives.
- 6- A numerical investigation was conducted considering a configuration of tailings dam that was constructed using the upstream method (Zardari, 2013) to investigate the effectiveness of the additives on the chosen tailings dam's stability.

### 1.4.1 Lab Testing Programme

To accomplish the aforementioned goal, a set of lab tests had been conducted, including;

- 1- ICP tests.
- 2- X-ray Fluorescence tests.
- 3- Sieve analysis tests.

- 4- Hydrometer tests.
- 5- Specific gravity tests.
- 6- Unit weight determination.
- 7- Water content determination.
- 8- Column sedimentation test.
- 9- Consolidation tests.
- 10- Unconfined compressive tests.
- 11- Direct shear tests.

The tailings tested included two different types. No general information was provided with them due to confidentiality reasons. Thus, their chemical compositions were evaluated to identify any source of hazard from them. They were also tested for geotechnical properties inside the geotechnical engineering laboratory of the University of Western Ontario.

## **1.5 Thesis organization**

This thesis is organized in 5 chapters as follows.

Chapter 1 describes the problem investigated and the organization of the thesis structure and addresses the objectives and methodology of this research

Chapter 2 literature review of tailings dams is presented. Methods of tailings dam construction used in practice are also discussed as well as merits and demerits of each type.

Chapter 3 presents the characteristics of the mine tailings investigated. The fundamental properties of the used additives are disclosed. The lab tests conducted on the treated and



untreated mine tailings are addressed and their results are presented and discussed. The samples used and the preparation methods adopted for each test is described and commented.

Chapter 4 discusses the used approach for slope stability enhancement. After that, the model used is presented and discussed.

Conclusions of research outcomes and the attained objectives are summarized in chapter 5, suggestions of further research are also stated.

# **Chapter2**

# **“Literature Review”**

### 2.1 Definition

Mine tailings are the mine waste or mine by-products, which result from the mining operations while the ore matter is extracted. Typically, ore concentration in the processed minerals ranges between 1-30% with the remaining being mine wastes (Villavicencio, 2013; Bjelkevick, 2005). As a result, tremendous amounts of mining materials become waste with its subsequent disposal constituting both financial burdens and environmental threats. Annually, millions of tons of mine tailings are disposed of. To exemplify, in Chile, a million tons of mine tailings is generated daily. Furthermore, the global mine tailings from the aluminum industry reached 75 million tons in 2012 (Rout et al, 2012). However, a considerable amount of the tailings is used in constructing tailings dams, which, in turn, reduces the need for conventional water retention dams and the associated high cost of borrow materials. A typical water retention dam is depicted in Figure2-1. Using such a dam can have the following advantages:

- A. Dam can be built to its full height prior to mine operation.
- B. Surveillance of the dam construction can be carried out easily.
- C. Dam faces can be protected against erosion.

However, the drawbacks associated with this type of dam include:

- I. Increased initial cost.
- II. Requires a source of borrow materials.

In addition, disposal cost, environmental and physical complications of tailings disposal still need to be addressed and are inescapable.

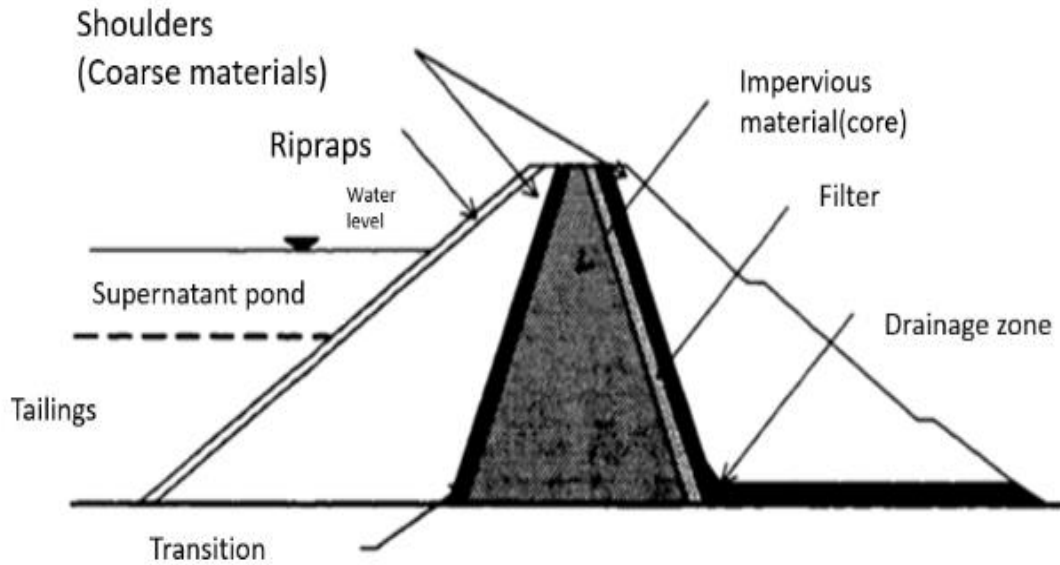


Figure 2-1 Typical cross-section of conventional water retention dam (After Priscu, 1999).

## 2.2 Risks

The vast area needed for waste disposal and the associated cost indicate the importance of sustained research efforts to identify efficient and reliable methods for their disposal. The cost of disposal will vary from one location to another and will depend mainly on the method of disposal employed and the distance to the dumping location. Nevertheless, eliminating the need for borrowed materials for the staged construction of the impoundment can reduce initial cost and required land (Priscu, 1999). However, because of their massive quantities and their random composition, mine tailings dams can have tremendous risk for the surrounding areas. This risk may be magnified due to seismic hazard, the weakness of the impounding materials, the toxicity of the retained wastes, the frequent occasions of heavy rainfalls, and the high phreatic level.

The main purpose of the enclosed pond method of construction is to retain the wet tailings, which are pumped into the progressively heightened impoundment(s). Thus, it offers cost effectiveness and ease of disposal; however, the saturation of those impoundments materials can

increase the risk of slope instability. There are various methods for impoundments construction. Economically, and to some extent environmentally, the preferred method is to use the mine wastes themselves in the embankments body in staged construction as the tailings will be disposed of at the same location, thus eliminating transportation cost. However, the use of materials that are strong enough to satisfy the stability requirements and meet the environmental restrictions is essential. As such, separation of tailings into coarse and fine materials is common to use the coarser stream in the construction of the dam's outer shell. The separation can be done by various measures such as spigotting, hydro-cyclones, and centrifuges. Accordingly, whether or not to exploit the mine tailings in the tailings dam's construction is dictated by the amount of sand yield. As far as the sand tailings embankments are concerned, three methods of staged construction exist: Upstream method, Downstream method and Centerline method. Those methods are discussed briefly in the following section.

## **2.3 Methods of Constructions**

### **2.3.1 Upstream Method**

This method involves raising dikes successively moving from the so called starter dike towards the center of the bond. It takes advantage of self-consolidating beach where the coarse particles settle close to the discharge spigot forming a beach while the finer materials flow away making slimes. In this construction method, two approaches are used, namely; mechanical and hydraulical deposition approaches (Jeyapalan, 1980; Priscu, 1999).

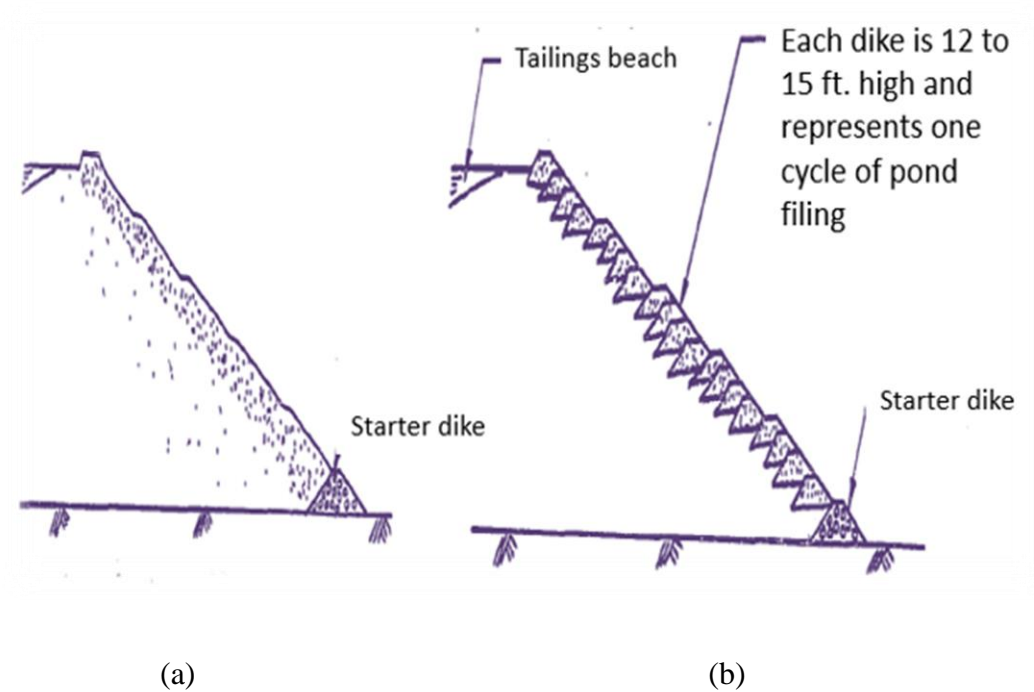
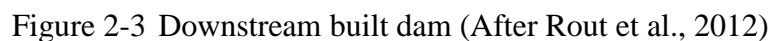


Figure 2-2 (a) hydraulically and (b) mechanically built upstream tailings dams  
(After Jeyapalan, 1980).

Both methods utilize pipelines for slurry transportation but the mechanically constructed dam (Figure 2-2b) has some merits over the hydraulically built dam in that the sand beach is compacted, resulting in a stronger outer shell that will promote dam stability.

### 2.3.2 Downstream Method

This method is safer than the upstream method though less used in practice, because its use is subject to the amount of sand yield and, most importantly, the geotechnical construction regulations (Jeyapalan, 1980; Priscu, 1999). For instance, after the disastrous earthquake event in 1965 that resulted in the liquefaction of El-Cobre mine, re-assessment of adopted construction practice of mine tailings was necessitated by the government authorities. According to the



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### 2.3.3 Centerline Method

Similar to the downstream method, the centerline method requires high sand yield. However, dikes are raised from both upstream and downstream sides of the starter dike, unlike the downstream method whose crest is shifted progressively towards the downstream side of the started dike. However, in the centerline method, both the fine and coarse materials are used in the construction of the upstream and downstream rises, respectively.

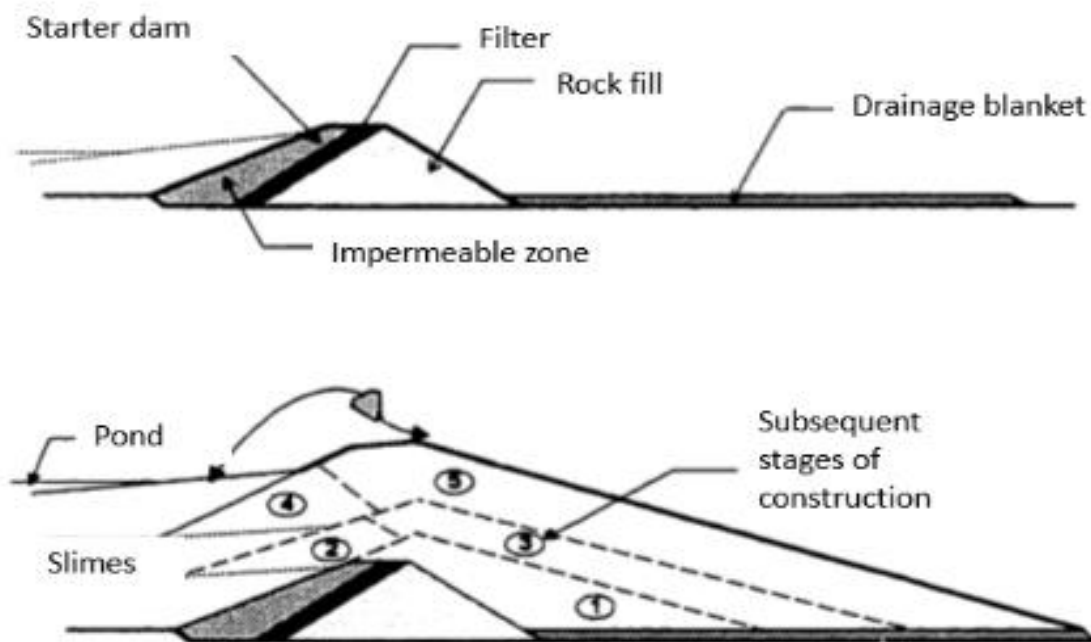


Figure 2-4 Centerline method construction sequence (After Priscu, 1999).

## 2.4 Pre-Dewatering

### 2.4.1 Thickeners

One way to produce an easy to reclaim soil layer is by utilizing thickeners. Thickeners operate as a vessel which employs a gravimetric sedimentation procedure where the slurry is poured into them and allowed to settle. Once settled, the thickened soil layer is removed as underflow and the expelled water is removed as overflow. This method allows much stiffer and



stronger, thereby easier to reclaim soil. It utilizes less water than natural dewatering method due to seepage and associated consolidation since the collected water can be further circulated to the mill and back to the sedimentation tank. Besides, coagulant can be used to expedite the rate at which particles settle.

### 2.4.2 Hydro-Cyclones

Hydro-cyclones are used to separate the coarse tailings, more than 0.075mm, and the fine tailings, less than 0.075mm. They operate by utilizing the centrifugal forces that act on the heavy soil particles when they enter from the top in a trajectory motion as a slurry to separate them from the fine particles. This happens as the slurry enters the hydro-cyclone in high speeds and spins around the inverted cone where the coarser particles are attracted to the cone-wall and the fine particles remain suspended in the water. The coarse particles make their way to the bottom where they are collected as underflow and the fine particles that remain in suspension exit from the top as overflow. The underflow is, however, not completely separated from fines. Once separated, the fine particles are either further processed for dewatering or pipeline transported to containment(s). The underflow is typically used for the outer shell of the impoundment which results in much stronger containment and a low hydraulic gradient; otherwise, they are stock-piled as a single or groups of piles which results in a robust embankment that requires less footprint.

### 2.4.3 Centrifuges

Centrifuges can be used to produce a “cake” of soil that is best suited for reclamation purposes. The centrifuge in this case is a rotating drum, which has two ends; one for inlet and the other for outlet for collecting free water. The wall of the centrifuge can be perforated and

covered with a filter for decanting purposes. When the slurry enters the centrifuge, the rotational motion drags the slurry to the wall whereby the centrifugal forces attracts the heavier soil particles and two distinct regions form, one is a “cake” of compressible soil where particles are almost fully in contact with each other and the other is free water zone. This will allow an efficient water circulation mechanism and drier materials. This material is then collected and can be transported by conveyors or trucks for stacking. This method is also suitable should usage of tailings outside the mine location be required (Beier, 2015).

### **2.5 Post Dewatering**

Post dewatering happens after the tailings are dumped into their permanent location. The tailings start draining their water instantly after they arrive to the final destination where they start transitioning from a viscous liquid into a soil medium. During this process, deposited tailings experience large strains. Typically, the coarser particles will settle close to the discharge point creating a beach while the finer particles travel a distance with the slurry into the pond where they settle making slimes. Once dumped, the soil particles begin settling and the soil-water interface lowers whereby two distinctive regions form; a settling zone and free water. With time, a cake is formed in which the lower part is of lower water content than the top (Kabwe et al., 2013). This cake transforms to become a layer with time, where its weight affects the consolidation process. As the deposition continues, this layer becomes several layers that are self-consolidating due to the successive loading, generation and dissipation of excess pore water pressure.

### 2.6 Shear Strength Parameters

#### 2.6.1 Anisotropy

Laboratory testing should be properly interpreted and must take into account the differences between field and lab conditions. The disturbance, moisture migration and stress relief should also be considered for samples retrieved from the field. Furthermore, the sampling and preparation methods should be appropriate for the analysis aimed. This truly holds when the analysis concerns facilities or structures built on top of layered strata, like varved clay, where deposition process has resulted in either stress or structural anisotropy (Chen et al., 2014). In mine tailings dams, this situation will inevitably exist as the mining deposits are being built successively. For slope stability problems, when the soil is tested for the shear strength parameters, caution should be exercised as the direction of shearing is neither horizontal nor vertical but rather curved. In this case, soil anisotropy should be considered in terms of which direction the samples should be sheared, horizontal (Direct Shear) or inclined (Triaxial Compression). For cases when testing an already deposited material for slope stability, the shape of the expected slip surface should be constructed utilizing the Finite Element method and this information employed to decide on which plane the field sample should be sheared. It is believed that triaxial compression will mostly provide the best results, and more conservative answers, about the shear strength parameters of most soils (Vick, 1990; Das, 2006). Chen et al. (2014) presented a series of lab testing conducted on direct shear box. They noticed that the shear strength parameters vary according to the inclination of bedding angle with a difference in the angle of internal friction up to  $7^\circ$  between  $90^\circ$  and  $0^\circ$  bedding angle.

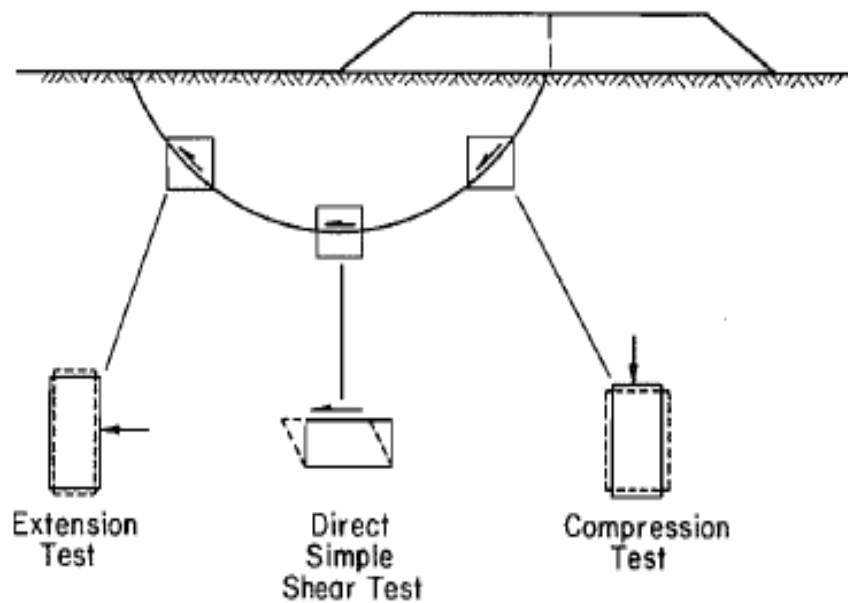


Figure 2-5 Arbitrary slip surface location and the different forms of shearing along it (After Mesri, 1989).

Figure 2-5 clearly shows the variation in shearing direction and the incompatibility of lab apparatus to replicate all of these forms at once.

### 2.6.2 Shear Strain Rate

The strain rate used in testing the specimen should correspond to the expected loading conditions. For example, if high shearing strain rate, such as 0.5mm/min, is used in DST with saturated clay, excess pore pressure will not have enough time to dissipate resulting in a reduced confinement on the sample. This, in turn, will violate the use of the effective strength envelope to get the cohesion and the internal friction angle since the second component of equation 2-1 is no longer fixed, unless excess pore pressure measurements are available. The following are the used equations to establish the soil shear strength parameters.

$$\tau = c' + [\sigma' \tan (\Phi')]\dots\dots\dots (2-1)$$

$$\tau = c + \sigma \tan (\Phi) \dots\dots\dots (2-2)$$

Equation (2-1) represents the shear strength envelope for drained analysis where the effective stress stays assumingly unchanged for the case of finite shear strain devices, for the entire time of one test. For normally consolidated clays and sand, the equation reduces to  $\tau = \sigma' \tan (\Phi')$ .

Equation (2-2) is the shear strength envelope of undrained analysis and is when excess pore pressure develops resulting in a reduction to the effective stress applied.  $C$  and  $\Phi$  in this case are the total strength parameters. In many cases, the right hand side of this equation is ignored and  $\tau$  becomes independent of the effective stress amounting to a single value represented by  $S_u$ , or undrained strength of the soil. This is depicted using Mohr-Coulomb criterion by the figure follows.

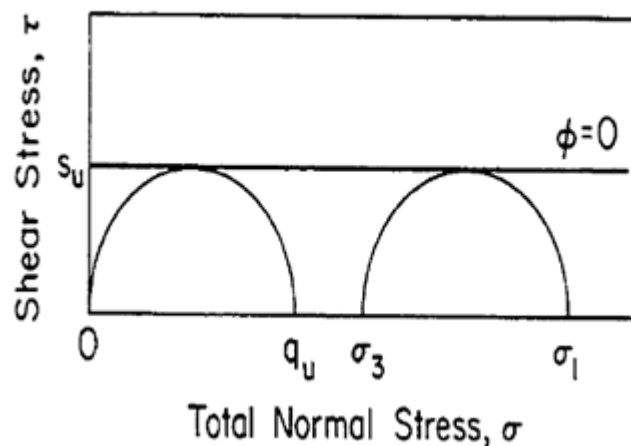


Figure 2-6 Mohr-Coulomb failure envelope for undrained cases.

However,  $S_u$  is not constant for all clayey soil regardless of their over-consolidation ratios. Mesri (1989) presented that the  $S_u$  can be generally estimated as follows;

$$S_u = 0.22 \sigma'_p \dots\dots\dots (2-3)$$

However, Mesri indicated that form of shearing, plasticity index, rate of shearing and the apparatus used can have significant impact on the results. This is consistent with the findings of Kulhawy and Mayne (1990) who showed that the rate of strain affects whether drained or undrained conditions exist and the shear strength of the material. Kulhawy and Mayne (1990) correlated  $S_u$  values and the strain rate. The correlation is depicted in Figure 2-7, which clearly shows how the undrained shear strength can be 30%± its value at 1% strain rate/hour.

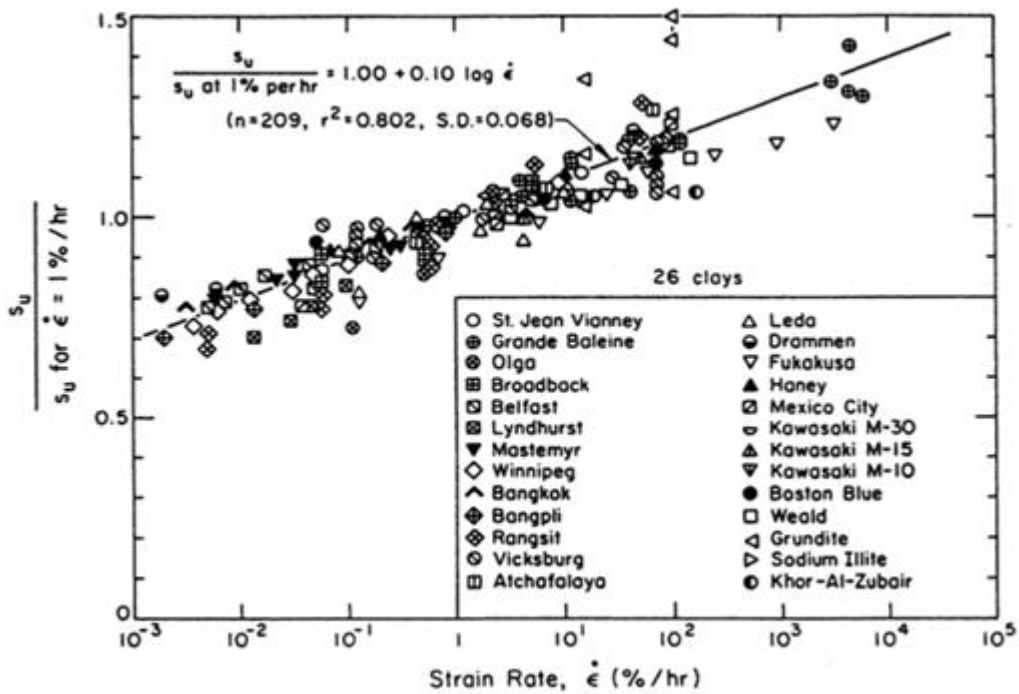


Figure 2-7  $S_u$  dependency on the shear strain rate (after Kulhawy and Mayne, 1990).

The relevance of this to the current study is that the shear strength parameters of the material used for stability analysis ought to represent the expected shear strain in the field. Higher strain

rates may show higher strength parameters whereby overestimating the stability of the assessed slope.

### 2.7 Consolidation

Soil consolidation is a time dependent process resulting from the expulsion of water from the soil pores. The consolidation settlement consists of primary consolidation and secondary compression or creep. Primary consolidation is the change in volume of a saturated fine-grained soil caused by the expulsion of water from the voids and the transfer of load from the excess pore water pressure to the soil particles. Secondary compression is the change in volume of a fine-grained soil caused by the adjustment of the soil fabric (internal structure) after primary consolidation has been completed. The primary consolidation time will therefore be dependent on the drainage properties; namely, the hydraulic conductivity and the boundary condition. The Oedometer test results are typically used to establish the coefficient of consolidation, which can be used to forecast the time necessary for a certain consolidation percentage to take place. The  $C_v$  can be estimated as follows;

$$C_v = \frac{K}{m_v \gamma_w} \dots\dots\dots (2-4)$$

where;

K: Hydraulic conductivity

$m_v$ : Coefficient of volume change

$\gamma_w$ : Water unit weight

The primary consolidation can be estimated as follows (Das, 2006);

$$- S_c = \frac{c_c * H}{1 + e_o} \log\left(\frac{\sigma'_{0} + \Delta\sigma'}{\sigma'_{0}}\right) \text{ (Normally consolidated deposits)} \dots\dots\dots (2-5)$$

$$- S_c = \frac{c_s * H}{1 + e_o} \log\left(\frac{\sigma'_{0} + \Delta\sigma}{\sigma'_{0}}\right) \text{ (Over-consolidated soil with stress increase plus existing overburden less than the pre-consolidation pressure)} \dots\dots\dots (2-6)$$

$$- S_c = \frac{c_s * H}{1 + e_o} \log\left(\frac{\sigma'_c}{\sigma'_{0}}\right) + S_c = \frac{c_c * H}{1 + e_o} \log\left(\frac{\sigma'_c + \Delta\sigma}{\sigma'_c}\right) \text{ (Over-consolidated soil with stress increase plus existing overburden more than the pre-consolidation pressure)} \dots\dots\dots (2-7)$$

$C_c$  and  $C_s$  are slopes of virgin consolidation line and rebound swelling (unloading line), respectively. They are estimated from the  $e$ - $\log \sigma'$  curve as follows;

$$- C_c = \frac{e_1 - e_2}{\log\left(\frac{\sigma_2}{\sigma_1}\right)} \dots\dots\dots (2-8)$$

$$- C_s = \frac{e_1 - e_2}{\log\left(\frac{\sigma_2}{\sigma_1}\right)} \dots\dots\dots (2-9)$$

The previous symbols used are defined as;

$S_c$ : Primary consolidation settlement

$\sigma'_0$ : Existing overburden

$\Delta\sigma'$ : Effective stress increase

$\sigma'_c$ : Pre-consolidation stress

$e_o$ : Initial void ratio

$H$ : Thickness of soil layer

Due to complex loading/pore pressure generation and the subsequent consolidation cycle observed in mine tailings dams, numerical analyses were used employing advanced soil models



to assist engineers to predict settlement and excess pore water pressure. These sophisticated numerical tools allow considering staged construction where time and boundary conditions present a major challenge. The two main constitutive models employed in analysis are Mohr-Coulomb and Modified Cam-Clay model. In Mohr Coulomb model, the Oedometric results are less valuable compared to the Modified Cam-Clay in the sense that it does not consider the e-log  $\sigma'$  curve (soil non-linearity) but rather use the typical stress-strain (simple elastic-plastic) relationship to account for settlement. However, it is seen to be of acceptable accuracy in calculating the settlement of such complex load/EPP generation/consolidation behaviour for a slope stability problem. In addition to its acceptable accuracy, it is simple and requires only five parameters, in addition to the unit weight, that are easy to estimate. Conventional Oedometer testing is not suited for slurried mine tailings where the strain level expected is large. What is also lacking in these conventional consolidometers are the ability to exert very low effective stresses, between 0.1- 1 kPa, on the slurry. Therefore, Gibson (1967) proposed the following equation to estimate self-weight settlement.

$$\frac{\partial}{\partial z} \left[ \phi \frac{d\sigma'_v}{de} \frac{\partial e}{\partial z} \right] + (G_s - 1) \frac{d}{de} \left( \frac{k}{1+e} \right) \frac{\partial e}{\partial z} = - \frac{\partial e}{\partial t} \quad \dots\dots\dots (2-10)$$

Large strain Oedometers have been fabricated and used since then. Nonetheless, the e-log  $\sigma'$  obtained from these large strain consolidometers defines an effective stress and void ratio beyond which the slope of virgin consolidation curve stabilizes. This is shown Figure 2-8. What is of importance to highlight is that this point on the e-log  $\sigma'$  curve from large consolidometer testing happens to lie within the small range of effective stresses, between 0.5-1.5 kPa. Additionally, the slope of the e-log  $\sigma'$  before the stabilizing point is dependent on the material and can be very flat (Kabwe et al., 2013) or very steep (Gan et al., 2013). Furthermore, the tests

can take weeks to finish (Gan et al., 2011). This method can be fitted to the soil of interest and thus can produce results of acceptable accuracy (Barnekow et al., 1999).

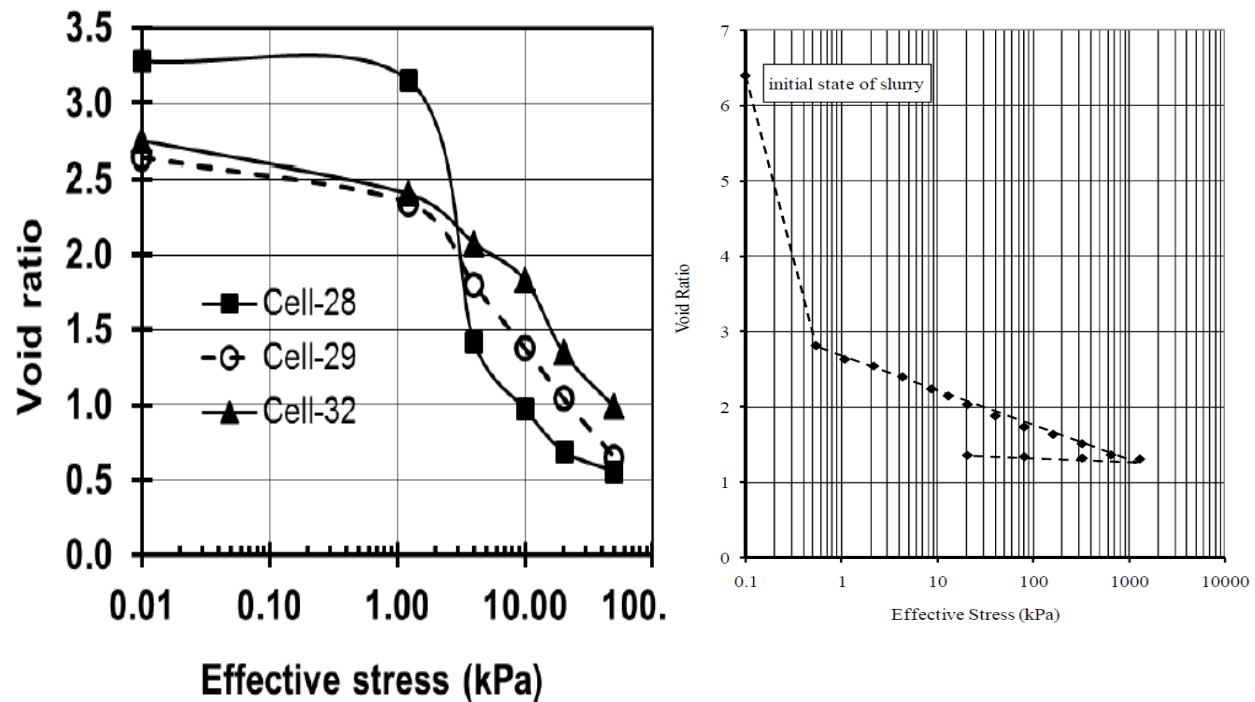


Figure 2-8 Large strain consolidation tests done on large strain consolidometers by Kabwe et al. (2013), left, and Gan et al. (2011), right.

The difference between the initial slopes can be attributed to water content and thixotropy.

Thixotropy is reported to add quasi over-consolidation pressure on the tailings. No settlement will take place until this pre-consolidation pressure is exceeded (Wilson et al., 2013).

### 2.7.1 Creep Settlement

Creep is the volumetric reduction of the soil volume at constant stress due to static pressures at the particle to particle contact that are in excess of what the soil particles can withstand. This causes soil particle rearrangement, which results in void ratio reduction. It is important to

consider this soil time-dependent behaviour if suspected, especially when planning mine tailings dam closure. This problem can lead to very problematic situation if the land is set to be used for future land development. Secondary consolidation index and Creep settlement can be estimated according to Das (2006) as follows:

$$C_{\alpha} = \frac{\Delta e}{\log\left(\frac{t_2}{t_1}\right)} \dots\dots\dots (2-11)$$

Where;

$C_{\alpha}$ : Secondary consolidation index

$\Delta e$ : Change in void ratio after primary consolidation phase ends

$t_1, t_2$ : Two arbitrary times after the end of primary consolidation

$$C_{\alpha}' = \frac{C_{\alpha}}{1 + e_p} \dots\dots\dots (2-12)$$

$$S_s = C_{\alpha}' H \log\left(\frac{t_2}{t_1}\right) \dots\dots\dots (2-13)$$

Where;

$S_s$ : Secondary settlement

H: Thickness of soil stratum

$e_p$ : Void ratio at the end of primary consolidation

### 2.7.2 Particle Crushing

Particles crushing and grinding cause tailings to become angular and sub angular in shape, which results in high contact pressures between particles that could lead to particle crushing and weakening. Therefore, it is important to study the effects of high stresses on tailings where soil time dependent behaviour is not well understood. In this thesis, however, this issue was not directly addressed as no long term results were studied to observe the effects of sustained load on creep and shear strength values. However, the effect of high stresses on the shear strength parameters was investigated.

## 2.8 Geotechnical Parameters of Tailings

The typical compression index,  $C_c$ , for mine tailings ranges from 0.05 to 0.87 (Qiu and Sego, 2001; Barnekow et al., 1999), and their specific gravity,  $G_s$ , varies from 2.65 to 3.97. Water content varies substantially from tailings to tailings with a range of 20-160% and depending on deposition method, liquid limit of the tailings material and if dewatering is used (Shuttle and Cuning, 2007; Mahmood and Mulligan, 2010; Qiu and Sego, 2001; Barnekow et al., 1999). The gradation of tailings depends on the parent rock and the degree of crushing and grinding to which tailings are exposed. A broad range of grain size can happen in one particular tailings, ranging from fine sand to clay. Figure 2-9 shows some examples for tailings gradation.

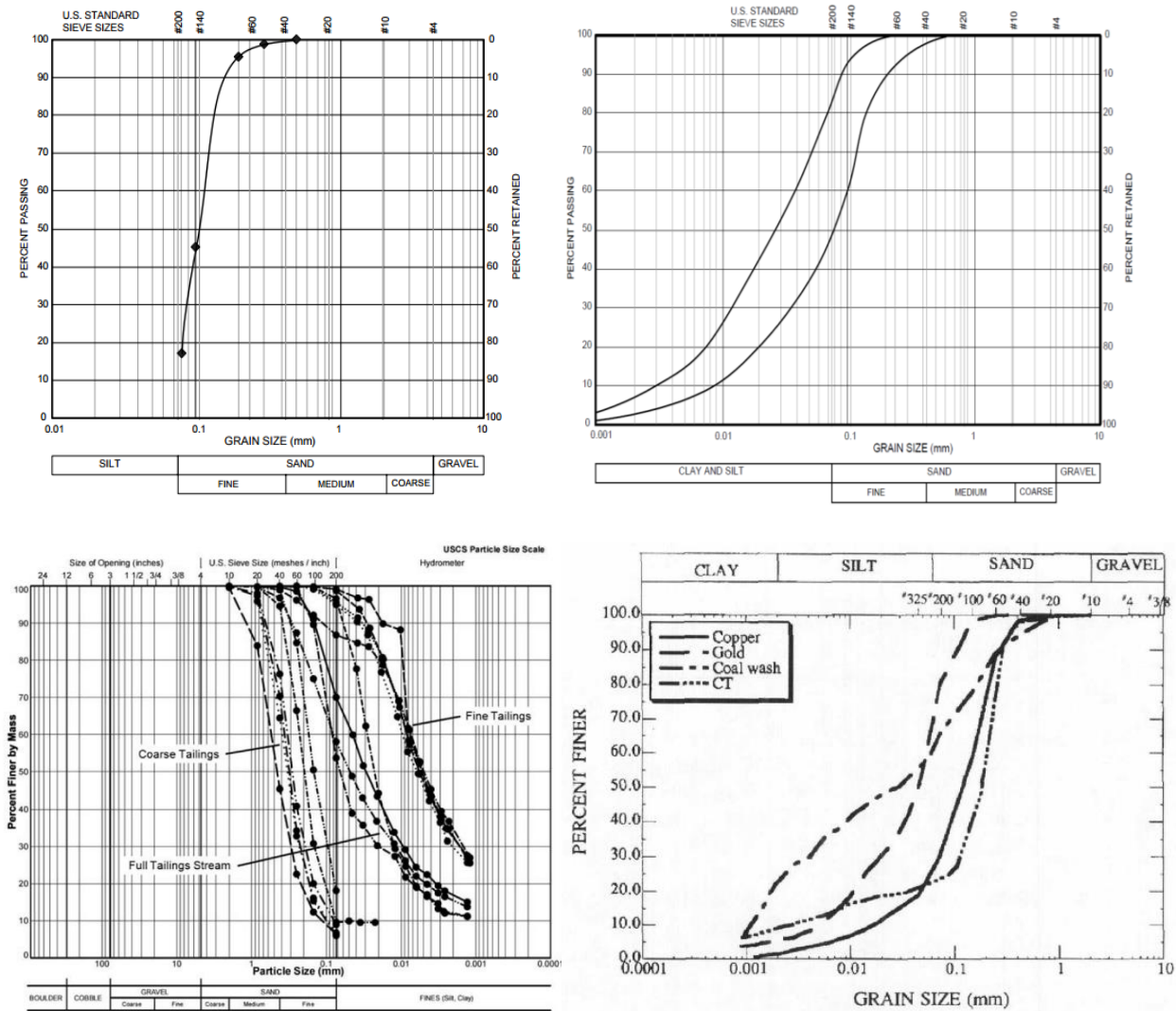


Figure 2-9 Wide range of tailings gradation (After James et al, 2002; Ozcan, 1986; Qiu and Sego, 2001).

### 2.8.1 Relative Density

Coarse tailings are found to have relative densities between 40-60% and can be classified accordingly as loose to compact materials (Kabwe et al., 2013; Anderson and Eldridge, 2011; Das, 2006; James et al., 2002). Table 2-1 shows soil classification according to their relative densities.

***Table 2-1 Soil classification according to relative densities.***

Dr, %	Description
0-20	Very loose
20-40	Loose
40-60	Compact
60-80	Dense
80-100	Very Dense

***Table 2-2 Typical friction angle range with relative density.***

N value	Friction Angle, $\phi'$ (Deg.)	Relative Density, $D_r$ (%)	Description
Less than 4	25 - 28	Less than 15	Very loose
4 - 10	29 - 32	15 - 60	Loose
10 - 30	33 - 35	60 - 75	Medium
30 - 50	36 - 40	75 - 90	Dense
Over 50	41 - 45	Over 90	Very dense

From this previous information, one can conclude that the angle of internal friction of such tailings would be between 33-35° (Bery and Saad, 2012). This conclusion agrees well with the range of internal friction angles reported in literature (Ozcan et al., 2012, Zardari, 2013).

### 2.8.2 Hydraulic Conductivity

The tailings gradation occupies the region between coarse sand and clay. Most of the tailings will have a combination of coarse and fine particles. Therefore, it is difficult to characterize the tailings hydraulic conductivity,  $k$ , due to several reasons such as the disposition method and history, gradation, stress history and so on. Thus, using a single correlation, such as Hazen equation, to get a single value for hydraulic conductivity is erroneous. For instance, Bjelkevik (2005) compiled data to compare the lab measured hydraulic conductivities of 7 Swedish tailings dams and those calculated using two equations, i.e. Hazen (Das, 2006) and Chapuis (Bjelkevik, 2005). Bjelkevik found that the ratio of calculated to estimated hydraulic conductivity varied between the two equations and noted that Chapuis equation overly overestimated the hydraulic conductivity. Though same overestimation was noticed in the case of Hazen's equation, a lesser ratio, calculated/measured ratios of  $k$  between 2-3, was observed. Nevertheless, it is observed that using this equation for a certain dam, and for all the sampling locations, had consistently either over or underestimate the hydraulic conductivity. Furthermore, the ratio between the calculated and estimated were consistent for the three sampling locations. However, the effects of sampling, stress relief and stress history were not addressed (Bjelkevik, 2005).

In sub-aerial deposition, the tailings transforms from slurry to suspension to a cake layer, and finally to a soil deposit under water. They then undergo consolidation due to the staged construction process, which results in changes in void ratio, porosity, and therefore hydraulic conductivity. These parameters will affect the hydraulic conductivity and the dam stability. Therefore, it is important to account for these effects on the hydraulic conductivity value used in the numerical model, which will dictate how much time is needed to reach a certain excess pore pressure at certain location in a given time. Figure 2-10 shows a typical  $e$ - $k$  relationship. Values

of  $k$  will therefore be dependent on the soil type and stress history. Typical range of  $k$  found in literature is between  $1 \times 10^{-3}$ – $1 \times 10^{-8}$  cm/sec. (Qiu and Sego, 2011; James et al., 2002).

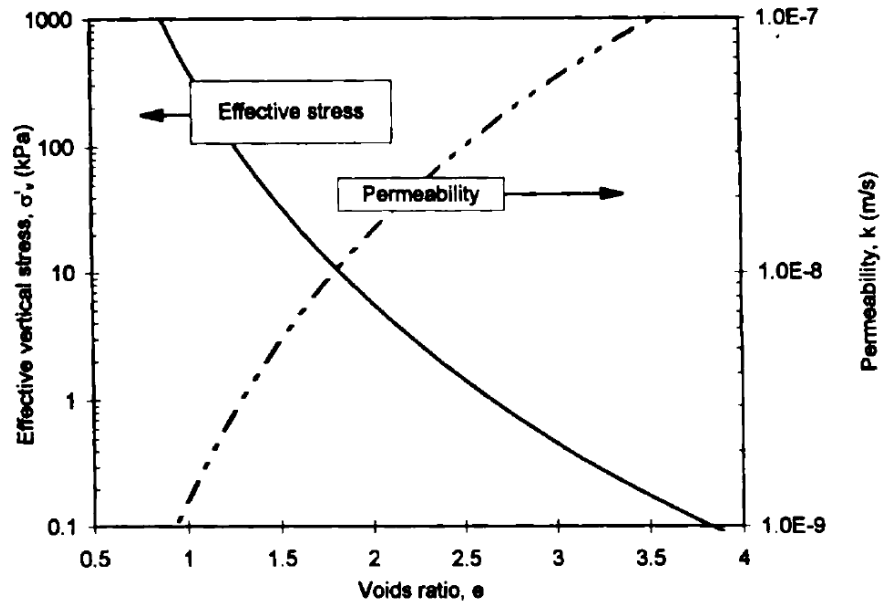


Figure 2-10 Decreasing  $k$  value with respect to void ratio (After Seneviratne et al., 1996).

### 2.8.3 Rate of Deposition and Dam Slope

There is no rule as to how the tailings dam should be heightened or what slope to use. It is always dependent on the operator choice which is affected by the available land, volume of tailings, method of deposition used, amount of sand yield and topography. It is reported that lifts of thicknesses in the range of 0.15-3m have been used to construct tailings dams (Fourie et al., 2011; Zardari, 2013). Slope inclination varies from 1:1.5 to 1:5, which also differs for upstream and downstream slopes. The upstream face of the tailings dam is usually built to a steeper angle than the downstream face (Anderson and Eldridge, 2011; Fourie et al., 2011).



### 2.9 Liquefaction Studies

Fourie et al. (2001) have reviewed the testing campaigns that were launched after the failure of the Merriespruit gold tailings dam in 1994. They pointed to static liquefaction as a possible explanation for the failure, which resulted in 17 casualties and 3km of mud and debris. The incident happened after a rainstorm of 50mm intensity, which caused overtopping and a breach in the northern wall of the upstream tailings dam. What is of interest is that the tailings disposal at that pond was halted a year before the incident took place due to high pond level and previous incidents of sloughing on the downstream face of the portion that later failed. It was mentioned that while South African tailings dams are allowed to maintain low freeboard, that tailings dam's freeboard had happened to be 200mm lesser than the minimum required freeboard of 500mm established by mining authorities in the county, the 1 in 100 years recurrence of a 24hours duration storm interval (Fourie et al.,2011). Fourie et al. (2011), have conducted a site investigation program supplemented by lab testing to test the hypothesis that static liquefaction was the reason of such catastrophe. Steady state line (SSL) was defined for materials with 60% fines and field void ratios were established from undisturbed samples and were plotted against the SSL where it was shown that 61% laid above the SSL which is an indicative of contractive behavior, or positive state parameter  $\psi$ . This review clearly demonstrated the importance of studying the nature of the tailings and their susceptibility to flow, complete loss of strength, or strain softening behavior, generation of EPP and reduction in effective stresses.



Figure 2-11 Aerial view of the breach that happened at Merriespuirt gold tailings dam (After Fourie et al.)

Sasitharan et al. (1994) conducted a consolidated drained triaxial test in which they have kept the deviatoric stress constant and decreased the mean effective stress in order to simulate a raised water table. They found that the sample failed very quickly for them to record data to the extent that the loading head had stuck the nuts causing excessive vibration and noise. This is consistent with the reported noise associated with the failure of Merriespuirt gold tailings dam, which is indicative of the tailings material having liquefied.

Similarly, Anderson and Eldridge (2011) reviewed the state of a 70 m thick slime deposit at an upstream tailings dam to assess the potential of slimes' susceptibility to liquefaction. The Chinese criteria for liquefaction assessment states that a soil deposit that contains more than 35% of fines will not liquefy (Dobry et al., 2001). This was established by post-earthquake

observations. This finding was attributed to the fact that the plastic fines hold the soil particles together. However, recent studies show that soil with fines up to 50% (silts), had liquefied (El-Takch, 2013; Boulanger and Idriss, 2006). Anderson and Eldrige (2011) conducted triaxial tests and considered the concept of critical-state soil mechanics to compare the in-situ void ratio with the critical state line (CSL). They found that, though the tailings were plastic, they have shown EPP built up and strain softening occurred, which indicate liquefaction can initiate. Fourie et al. (2001) reported a case study for liquefaction of a tailings dam. Figure 2-12 shows the CPTu soundings of the case study where the  $u_2$  measurements are depicted, while Figure 2-13 displays the results of laboratory testing of undisturbed tailings samples from the failure's vicinity.

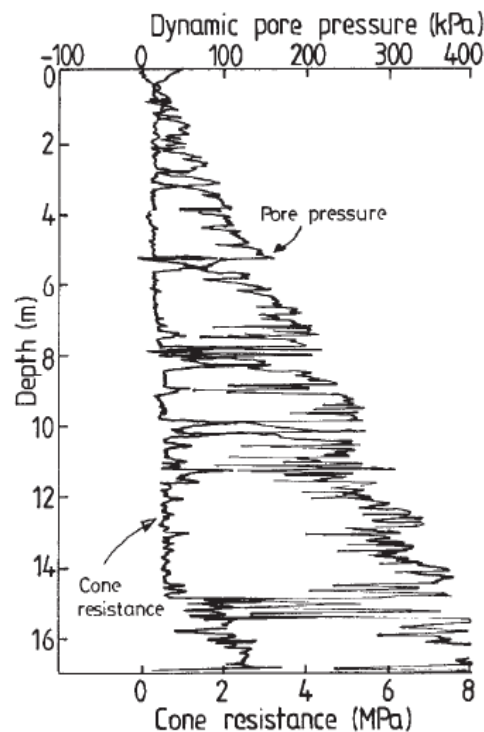


Figure 2-12 Piezocone results of  $Q_{tip}$  and  $u_2$  pore pressure measurements (After Fourie et al. 2001).

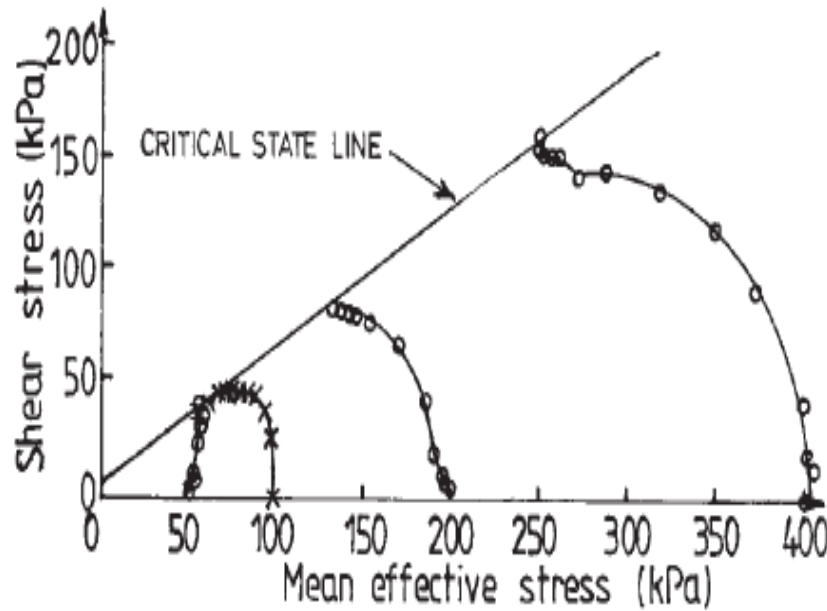


Figure 2-13 Aftermath laboratory testing of undisturbed tailings samples from the failure's vicinity (After Fourie et al., 2001).

It is clear from Figures 2-12 and 2-13 that the tailings were very soft material, where the cone tip resistance profile ( $Q_{tip}$ ) was almost constant throughout the top 10 m, within depth where liquefaction has been observed, with a value of approximately 500 kPa which is very low. Also, the dynamic pore water measurements,  $u_2$  indicate positive sign (contractive behaviour), with values of up to 300 kPa being recorded. The consolidated undrained (CU) tests results presented in Figure 2-13 demonstrate that the tailings displayed a contractive behavior, reduction in effective stresses and increase in positive pore pressure as shearing continues until CSL is reached.

Similarly, the CPTu and CU tests of the gold tailings dam (Anderson and Eldridge, 2001) demonstrated liquefaction behavior. As Figure 2-14 shows, the profile shows 5 m of sandy soil crust with  $Q_{tip}$  around 2 MPa and  $u_2$  is zero (i.e. drained behavior). This is underlain by 20 m of

silt overlying clay that extends to the bottom of the sounding. The positive  $u_2$  measurements and low  $Q_{tip}$  value below 40 m, which indicate contractive behavior of the tailings.

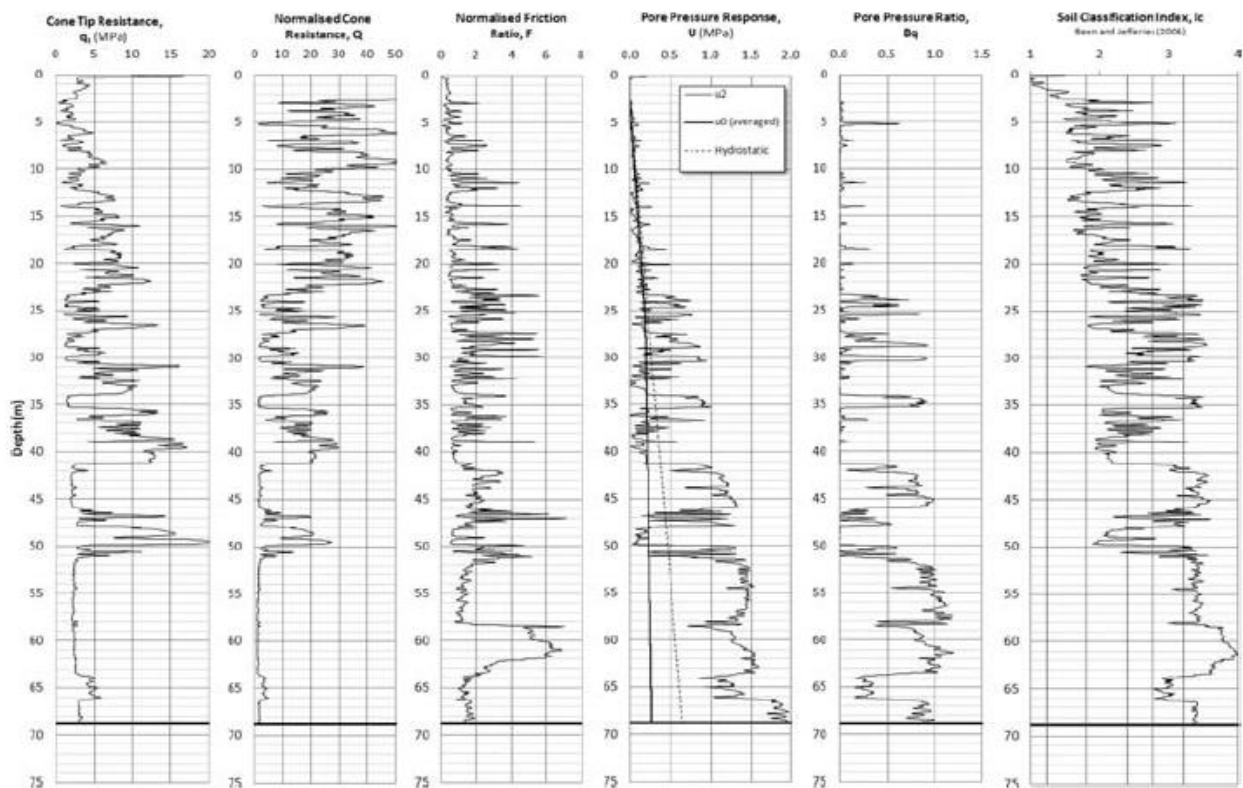


Figure 2-14 CPTu profile of upstream gold tailings dam assessed for liquefaction by Anderson and Eldridge (2001).

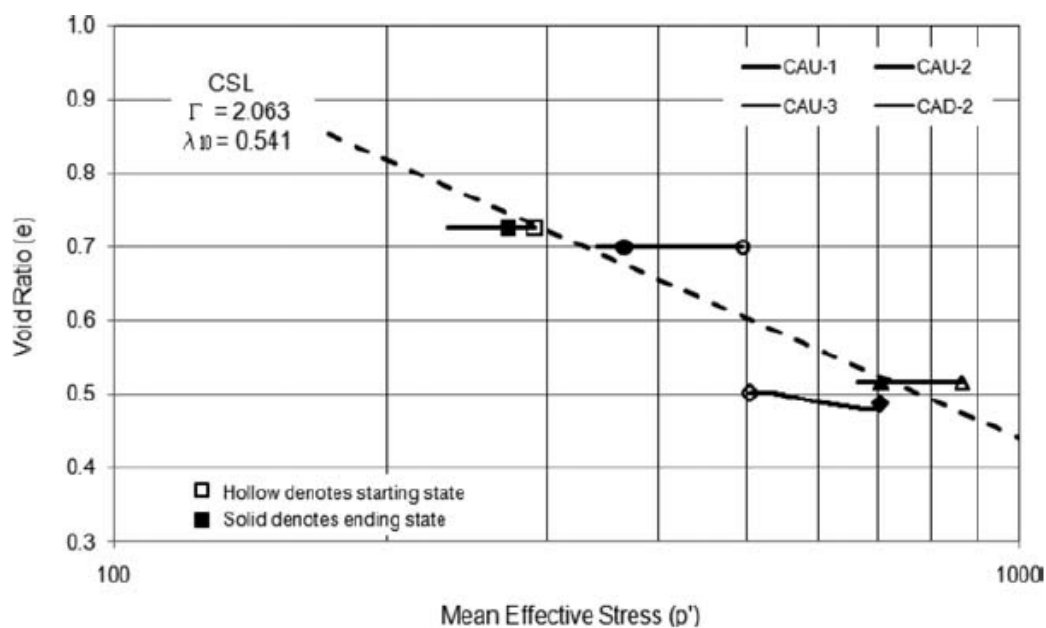


Figure 2-15 Contractive behavior for some of the CU tested tailings (After Anderson and Eldridge 2001).

James et al. (2003) have considered the drawbacks of small conventional testing such as small sample size and vertical applied stresses. They built a one cubic meter box that goes on a shaking table to test the effects of overburden, drainage path(s) and heterogeneity on the soil response. The oscillation produced by the shaking table they used can go up to 3g with the ability to reproduce any earthquake time history. They have not yet, up to the author knowledge, published all cases but a preliminary case. The preliminary case they published contains evidence of a bearing capacity failure at the ground surface which is something James et al. (2003) attributed to liquefaction, even though the EPP was not measured at the zone of collapse and the other pore pressure cells did not show EPP high enough to produce zero effective stress. The authors' conclusion about that is that the small effective stresses, mean of 2.5kPa, helped the soil mass not to soften and generate liquefaction except at the ground surface where there was a surcharge of 2.5kPa.

It may be concluded from these cases that mine tailings are susceptible to strain softening and contractive behaviours, where positive EPP can generate to the extent of soil structure collapse as the effective stress approaches zero. They help to apprehend the undrained tailings behavior, which can put the tailings dams' safety at risk. This has to be addressed in advance of construction because EPP can develop in any tailings dam due to the staged construction practice associated with the wet deposition method, which could lead to static liquefaction and/or reduced effective stresses thereby reduced safety factor of the tailings dam whereby putting the stability of the whole TSF and the surrounding environment at severe risk.

In order to properly account for these issues in this thesis, an effective stress analysis (ESA) approach was adopted in the numerical analysis phase to evaluate the influence of staged construction on the stability of the tailings dam. ESA analysis allows realistic modeling of the observed field behavior. Rourke et al. (2013) presented a liquefaction assessment using CPTu and two available methods, first by Robertson et al. (2009), uses soil behavior index ( $I_c$ ) and correction factor ( $K_c$ ) to account for fines, and second by Idriss and Boulanger, developed for sand (2008). What was noticed is that the two methods provided S.F against liquefaction that are inconsistent with each other. The recommendation Rourke et al. (2013) had made is more research on this grey zone to improve assessment guidelines. Generally, the cyclic resistance ratio (CRR) will increase with the increase of fine contents and the decrease of void ratio. Furthermore, it will increase with increasing the PI which is seen to be linked to the fine contents. It is not clear whether this increase is due the reduction in SPT value by the presence of the fine contents or is due to the fine action as a retarder to soil structure collapse (Dobry et al., 2001).

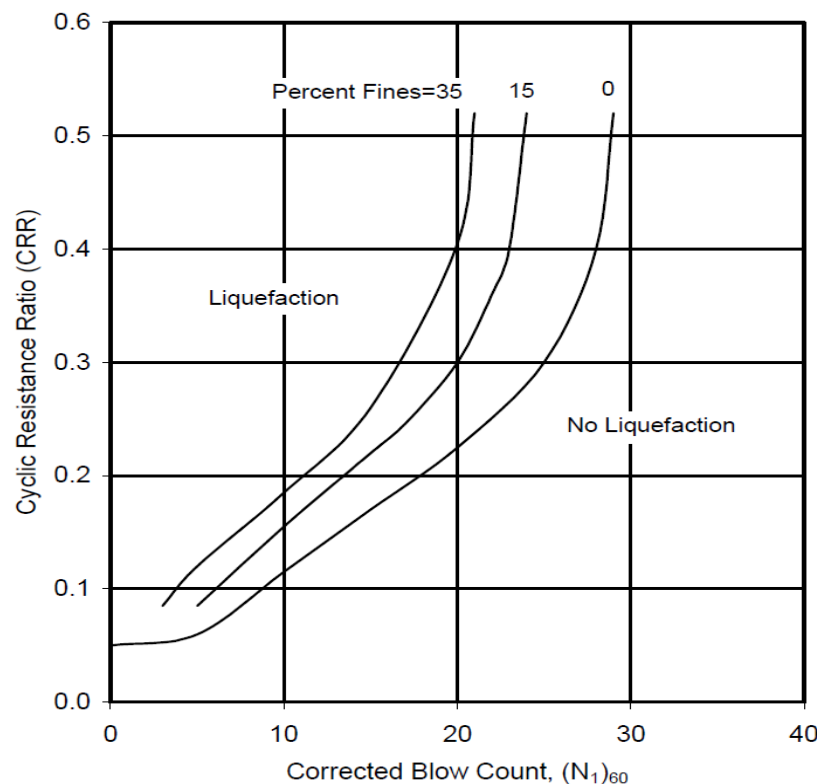
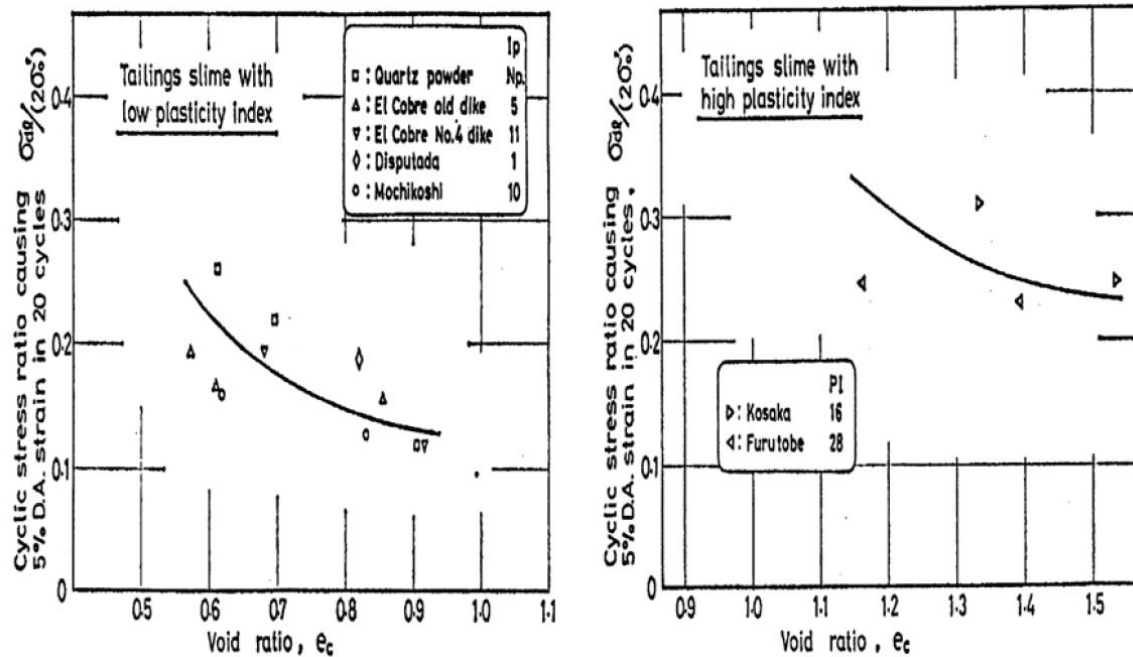




Figure 2-16 Effect of fine content on the CRR value (James et al., 2002).



(a) Low plasticity slims

(b) High PI slims

Figure 2-17 Effect of void ratio and PI on the CRR value (Zardari, 2013).

## 2.10 Review of Published Numerical Models

Advanced numerical programs can be very useful in estimating important engineering parameters related to both the design and construction. These programs (e.g. PLAXIS 2D) offer user friendly tools that are easy to implement and also allow adaptation to the specific problem analyzed. There are a few numerical investigations of tailings dams reported in the literature. For example, Ozcan et al. (2012) studied the downstream slope stability of a Turkish tailings dam when its storage capacity was increased. They utilized the commercial 2D program Slide V.5.0, which uses limit equilibrium method for slope stability analysis. They found that, due to raising



the crest, the dam safety factor increased from just below 1.2 to 1.58, which is attributed to the fact that the addition of the new raise contributed to a flatter slope. However, when the phreatic level was increased, the safety factor dropped to 1.12, below the recommended CANMET (1977) regulation which requires that a minimum safety factor for downstream tailings to be no less than 1.2 (Ozcan et al., 2012).

Ormann et al. (2013) investigated the stability of Aitik tailings dam, Sweden, to evaluate its stability after a new proposed raising sequence was proposed. They studied the effect of the staged construction on the safety factor using the 2D commercial program PLAXIS. They concluded that the safety is affected by the cycle of loading/EPP generation and the associated consolidation and proposed a new method of stabilization using rock-fills. Yin et al. (2011) analyzed the slope stability of a prototype scale tailings dam using a laboratory physical model (1: 200 scale). They investigated the influence of staged construction of the dam employing Slide V.5.0. They have used the phreatic level of the lab model to establish the static safety factors, under both normal and flood situations. They conducted a dynamic analysis in which the phreatic level in the physical model was updated to the level established from the numerical model. They concluded that the safety factor under normal condition was the highest. When flood and dynamic excitation were modelled, a significant drop was observed. They have also recommended the dam's height be limited to 100m. Seneviratne et al. (1996) extended the work of Gibson (1967) and developed the consolidation program "MinTaCo" for the purpose of estimating the self-consolidation settlement of tailings dams. They estimated the self-weight consolidation at different rates of filling and evaporation. Their program can be used to optimize the available storage by taking into consideration the evaporation and filling rates. Similar work was done by Beier et al. (2015) using the program TMsim, which is coupled with Excel macros

and FSConsol. Barnekow et al. (1999) have used FSConsol program to study the consolidation settlement of WISMUT tailings dam, which was constructed using the centerline method. They investigated the excessive self-weight settlement, amounting to 0.2m per year for an average thickness of 45m. Characterizing the compressibility of the tailings slimes and knowledge of filling history are necessary to define the final shape of the dam in order to estimate the capping layer thickness. They evaluated variation of  $e$  with depth and verified the results by means of field testing, which confirmed the program accuracy.

### **2.11 Ground Improvement**

Ground improvement can be accomplished through physical, chemical, biochemical, dynamic processes in order to enhance ground engineering properties (Naeini et al., 2012). In this thesis, the ground improvement term will be substituted by soil improvement as it is more specific and related to the research goals. Employing additives as binders to the soil mass is one of the methods by which the soil is stabilized and improved. The additives strengthen and stiffen the soil by binding the soil matrix by cohesive forces. Various techniques are used in practice, such as deep soil mixing, jet grouting, lime and cement injection, slurry trench technique, stone and sand columns, static and dynamic compaction and wick drains. The aim of these techniques is to solidify or densify the soil mass yielding a more competent stratum. Furthermore, soil is reinforced when composite soil-geosynthetics mass is formed by certain arrangements and alignment of reinforcing materials into the soil volume as in the case of geogrids and geotextile. These methods are widely adopted in engineering practice and the selection of the appropriate method is based on several factors. Among these factors are: safety, cost, effectiveness, availability, adequacy and durability. In order to strengthen and stiffen the soil in its natural state, spraying, jet grouting, and soil nailing methods are preferred. However, these techniques are not

suitable for the case of tailings dams owing to their construction method and/or to the limited accessibility to project site.

### 2.11.1 Available Approaches to Slope Stabilization

Nikbakhtan et al. (2007) presented a case study of the efficacy of jet grouting in increasing the slope stability. Their parametric study was done using the finite element program CLARA-W. They compared the factor of safety in the staged construction of the Shahriar Dam, Iran using 2D and 3D analyses. Shu-wei et al. (2013) and Poulos (1995) examined the usefulness of micro-piles and piles in increasing slopes' stability. For the mine tailings dams built using the wet method, consolidated tailings, or composite tailings, (CT) and rock-fills have been used to increase the whole dam's stability. However, no attempt, to the author knowledge, has been done to "mix" the wet tailings with additives before disposal in order to strengthen the tailings.

In the current research, the emulsified polymers' compelling performance with different soil types stimulated their selection to stabilize mine tailings impoundments built using the upstream method of construction. Besides, traditional additive admixture of recycled Gypsum and Cement Kiln Dust (CKD) were also used to compare the two sets of additives seeking to identify the optimum additive.

### 2.11.2 Traditional and non-Traditional Additives

Quite extensive research is available in the literature on the use of traditional admixtures in order to strengthen soils. However, limited research is found on the use of non-traditional additives. The list of traditional admixtures includes: Cement, Lime, Fly ash, Cement Kiln Dust, Slag furnace, and Gypsum (Chapman et al., 2010; Jessie et al., 1977; Arioglu et al., 1986; Zou et al., 2004). Some researchers have used traditional and non-traditional additives in one admixture

to enhance the soil properties (Ates, 2013), while others have used non-traditional additives such as those derived from polymers in solid state (e.g. fibers, grids, and textiles; Alsunaidi and Ali, 2006; Mekkiyah, 2013) and liquid state (e.g. polymeric emulsions, Newman and Tingle, 2004). The common goal in all studies is to strengthen the soil. Whether this improvement is to meet serviceability limit state or ultimate limit state, the engineer should cautiously choose the optimum additive without compromising the cost and ease of implementation.

Solid polymer derivatives have recently gained wide popularity, as some were proven efficient in installation and performance and are cost efficient, with no to little observed environmental threats (Alsunaidi and Ali, 2006). Many studies were conducted on soils reinforced by solid polymer materials and by traditional admixtures. For instance, Mekkiyah (2013) investigated the improvement in bearing capacity of sandy deposits strengthened with solid polymer fibers underneath square footings. His results proved that bearing capacity was drastically enhanced (up to 800% increase in bearing capacity), with the mixing of solid polymer fibres. Alsinadi and Ali (2006) studied the effects of geo-grids on the bearing capacity of isolated square footings for public school project situated on very weak engineering fill whose unimproved bearing capacity was only  $0.6\text{kg/cm}^2$  and, similar to Mekkiyah's results, found that the bearing capacity increased by a factor of eight. Other researchers used geogrids (Liu et al., 2012), geotextiles (Hu et al., 2011) and fibers (Mekkiyah, 2013). However, these results are associated with the local soil conditions and should not be extrapolated to another situation.

Although both traditional and non-traditional additive are used in practice, scarce research is available in the literature on the polymeric emulsions and their potential use in soil improvement (Newman and Tingle, 2004; Ates, 2013; Naeini et al, 2011). Thus, they were under scrutiny in this research in order to investigate their potential use. Bench scale testing on reconstituted

tailings sand and non-tailings sand produced promising results in terms of non-traditional additives being potentially candidates for future engineering projects. Amongst non-traditional additives, a commercial emulsified polymer was chosen because some researchers reported improvement in both strength and stiffness when using it (Ateş, 2013; Lightsey, 1969; Naeini et al., 2012).

Acrylic polymers and biopolymers inclusion can be used to increase resistance to erosion, ductility and water retention (Haung and Liu, 2012; Chen et al., 2013; Orts et al., 1999; Maghchiche et al., 2010). Polymers infusion in an expansive soil in lab model is reported to have reduced the soil erosion and expansive and shrinkage potentials (Huang and Liu, 2012). Also, employing polymeric emulsions as a sand erosion countermeasure is further stated to have been implemented in many countries across the continents (Huang and Liu, 2012). Others have compared soil stabilization with polymers mixed with different materials (Newman and Tingle, 2004) while some have added natural waste materials such as palm kernel, and wool with polymer's derivatives (Marín et al., 2010; Khan, 2005). Those results suggested potential use of polymeric additives, either solid or liquid, in tailings facilities with the aim of strength and stability increase.

Different admixtures are attempted to conform to two aspects: shear strength and environmental impact. The latter could be met by using a recycled or waste material(s), which leads to reducing the cost of borrowing and satisfies environmental issues (given it does not leach any pollutant). Reducing the cost, however, is tricky.

### 2.12 Closing Remarks

The state of practice in construction of tailings dams was reviewed. The upstream method was found to be the most widely used due to its relative economic advantage, effectiveness and for site specific issues such as ruggedness and limited accessibility to the dumping location. For example, in China alone, it is reported that the number of tailings impoundment are in the order of 12000 where 95% of them are built using the upstream method (Yin et al., 2001). However, the upstream construction method has been the highest in number of reported incidents (Rico et al, 2008). Though, for its merits, the author is inclined to uphold exploiting it with some improvement. Therefore, the efficacy and cost effectiveness of the upstream method are incentive for the author to investigate this method thoroughly. Finally, using soil improvement technique by mixing the tailings with some additives that stiffen and strengthen the soil is proposed.

# Chapter 3

## **“Laboratory Testing”**

### 3.1 Introduction

This chapter reports on the experimental phase of the study. It covers both the initial characterization of the tailings materials used as well as the test results of the tailings material treated with different additives to improve its stiffness and strength. Tailings tested were two types, denoted tailings1 and tailings2, received from Golder Associates from mines in Northern Ontario. Tailings2 was only used for optimization work with traditional additives utilizing UCS machine. The rest of the tests are conducted with tailings1. This was done because tailings1 was in small in quantity to do optimization work. Several tests were conducted to characterize the tailings material that was used in the experimental program as well as the additives used for strengthening of tailings. The tests included chemical analysis and geotechnical tests for evaluating physical and mechanical properties. In addition, comprehensive testing was conducted for the treated tailings material and the obtained physical and mechanical properties were used in the subsequent numerical modeling to evaluate the effect of the treatment on the enhancement of the stability of tailings dams. In the following sections, the details of the experimental testing program are reported and the obtained results are presented and discussed.

### 3.2 Chemical Analysis

#### 3.2.1 ICP Test

Inductively Coupled Plasma (ICP) and X-Ray Fluorescence (XRF) analyses were conducted on the received tailings to define their chemical composition and identify sources of hazardous material in their composition. The ICP test can detect heavy metals such as (Cr, Ag, Fe, Co, Cd, Pb, Bi, Ba, V, As, Ni, Cu, Mn, Al, Zn, etc) and Alkali elements like (Na, Mg, Ca, K). It uses wave lengths of the material tested to match it with the spectrum of elements of interest. The device used is supported with the lists of wavelengths of the elements and minerals looked for.



For each element whose presence is to be detected, five wavelengths were chosen to enhance better accuracy of detection. The following chart reveals the detected elements and minerals within the tailings liquid. It should be noted that the heavy metal elements detected can pose a variety of health problems to humans in addition to being harmful to the environment in which we live.

Figure 3-1 presents the results of the ICP test conducted on samples of the tailings material used in the testing program. It is noted from Figure 3-1 that the tailings liquid does have a broad range of materials; however, this range of detected elements is not evenly present within the liquid. Rather, the dominant materials are those found in most drinkable water, which are potassium and calcium (1264 and 447 mg/L, respectively).

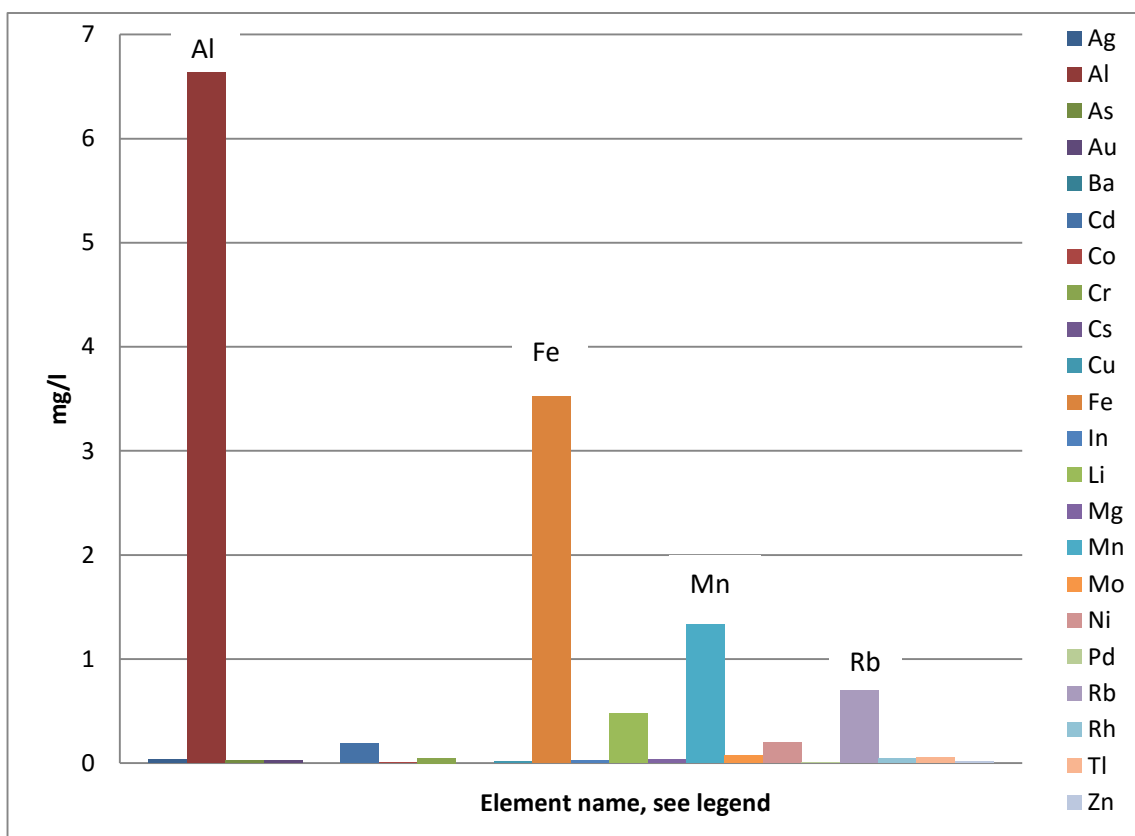


Figure 3-1 ICP test's results.

### 3.2.2 X-Ray Fluorescence Tests

This test was performed in order to identify the general chemical composition of the materials used. It was conducted at the Geo-chemical analysis laboratory at Western University. The method used herein is the pressed bullet method because it is fast and can screen up to 72 major elements. The tests were conducted on two types of tailings, as well as recycled gypsum and plaster that were used for treatment of tailings. Table 3-1 summarizes the results of this test.

*Table 3-1 XRF analysis results*

13.72	Recycled gypsum		Plaster		Tailings1		Tailings2	
	Wt%	StdErr	Wt%	StdErr	Wt%	StdErr	Wt%	StdErr
CaO	53.7	0.25	67.48	0.23	44.05	0.25	13.72	0.17
SO3	44.91	0.25	26.7	0.22	1.72	0.07	1.34	0.06
SiO2	0.543	0.027	1.82	0.07	32.38	0.23	44.63	0.25
Fe2O3	0.268	0.013	0.133	0.007	1.88	0.07	18.13	0.19
Al2O3	0.192	0.019	0.591	0.029	10.81	0.16	10.82	0.16
MgO	0.168	0.008	2.48	0.08	4.2	0.1	3.48	0.09
K2O	0.0576	0.0029	0.371	0.018	2.36	0.08	5.01	0.11
TiO2	0.0463	0.0041	0.0482	0.0046	0.822	0.041	0.58	0.029
Sc	0.0287	0.0027	0.034	0.003	0.0201	0.0024	0	0
La	0.0243	0.0058	0.0344	0.0068	0.0187	0.0058	0.04	0.0049

### Chapter 3: Laboratory Testing

Sr	0.0228	0.0012	0	0	0.068	0.0034	0	0
Na <sub>2</sub> O	0	0	0	0	1.07	0.05	0.86	0.043
MnO	0	0	0	0	0.258	0.013	0.17	0.008
P <sub>2</sub> O <sub>5</sub>	0	0	0	0	0.1	0.005	0.07	0.0033
Cl	0	0	0.0172	0.0024	0.0421	0.0024	0.7	0.035
Zr	0	0	0	0	0.0264	0.0017	0.01	0.0009
Br	0	0	0.0882	0.0026	0.035	0.0011	0.06	0.0019
Cu	0	0	0.0113	0.0017	0.0195	0.0015	0.03	0.0014
Ba	0	0	0	0	0.0194	0.0035	0	0
Pb	0	0	0	0	0.0176	0.0044	0	0
Cr	0	0	0	0	0.0096	0.001	0	0
As	0	0	0.0246	0.0079	0	0	0.04	0.0064
SrO	0	0	0.129	0.006	0	0	0	0
Sm	0	0	0	0	0	0	0.02	0.0024
Rb	0	0	0	0	0	0	0.016	0.008
Er	0	0	0	0	0	0	0.015	0.0012
Yb	0	0	0	0	0	0	0.07	0.0044
Dy	0	0	0	0	0	0	0.01	0.0014

Ho	0	0	0	0	0	0	0.04	0.0022
Tb	0	0	0	0	0	0	0.02	0.0021

Table 3-1 shows that all tested materials are rich in calcium; however, the plaster was richer than recycled gypsum (67.4 and 53.7% by total weight, respectively), which indicates plaster is more likely to offer increased strength to the treated tailings. Moreover, both types of gypsum are rich in sulfur trioxide, which is harmful to the environment as they can contribute to acid rain.

Furthermore, it is seen that the tailings are rich on Silicon oxide and Aluminum, which are light materials. The measured specific gravity of tailings<sup>1</sup> was 2.69, which lies within the range of silty clay materials; however, many tailings are reported to have higher specific gravity range due to the existence of heavy metals in them, which was not the case for these tailings. It was also noted that Arsenic, which is a hazardous material, was found in the plaster in high concentration, 0.024g/1000gram, which is higher than 0.01mg/L specified as Maximum Acceptable Limits (MAC) in Canadian Drinking Water guideline.

### 3.2.3 Significance of Results

Calcium and Potassium were found in high concentration. Thus, they were removed from the chart presented in Figure 3-1 in order to allow the graphical representation of the small range fluctuation of materials presence in the tailings liquid. Of significant, was the presence of some hazardous materials, which influence human beings adversely. These materials include: Arsenic, Cadmium, Chromium, and lead. Their concentrations in tailings<sup>1</sup> liquid were found to be higher than of those Maximum Acceptable Limits (MAC) found in Canadian Drinking Water Guidelines (CDW) by factors of 2.4, 37.54, 1.005, and 1.063. Around 100,000 gallons spill of

cyanide-contaminated water and tailings has been reported to migrate from a gold mine tailings pond in one incident happened Romania to nearby rivers believed to be source of drinking water to more than 2.5 million people (Davis, 2002). Accordingly, it is of utmost importance to have a robust lining and filtration systems around the TSF to ensure that no leaks, concentrated flow of water due to existence of fissures or cracks that create preferred path for water flow, or seepage, spatial flow of water due to difference in energy level to the ground water. It is also of more importance to maintain a robust tailings impoundment to safeguard the surrounding environment from any slope instability. Special care is usually followed to minimize or completely halt the flow of water to the ground water reservoirs. This sometimes creates a high ponded water volume, especially in cases where Acid Mine Drainage (AMD) issue is encountered. In such case, the tailings are sometimes maintained below the water level to reduce their exposure to oxygen in the atmosphere.

### 3.3 Sedimentation Column

As noted earlier, the emphasis in this thesis is on upstream tailings method of deposition. This means that the soil behaviour needs to be investigated and defined from its first phase, the liquid phase, up to its plastic state after consolidation. For the liquid phase, only the time to effective stresses formation is of interest for samples preparation's reason. This is because when knowing how tailings behave in a larger scale, then laboratory testing can be done in the best way to mimic the field condition. Rozalina and Yanful (2011) and Gan et al. (2011) discussed the concept of effective stress initiation after sedimentation, thereby initial void ratio at zero effective stress is achieved. Indeed, the sedimentation and consolidation processes continues until all excess pore pressure is dissipated and the soil grains become in contact with each other all the way through the bottom boundary.

This concept was verified in the current study by pouring the tailings, which were in slurry state, into a graduated cylinder 6 cm in diameter and 45 cm high. The slurry was poured to a level of 30 cm and the sedimentation process was monitored until the soil top surface level has not changed for a week. The results of this test demonstrate qualitatively and to some extent quantitatively how long would a tailings deposit take to change from slurry state into a soil stratum ponded by water from top. This is exactly analogous to field condition where, due to tailings deposits being successively placed on top of each other, the subsequent sedimentation and consolidation phases exist. It should be noted that the time for full sedimentation will vary, sometimes substantially, from one tailings material to another depending on the amount of fines, chemical composition, specific gravity, and electrical forces. Figure 3-2 depicts the interface level change versus time.

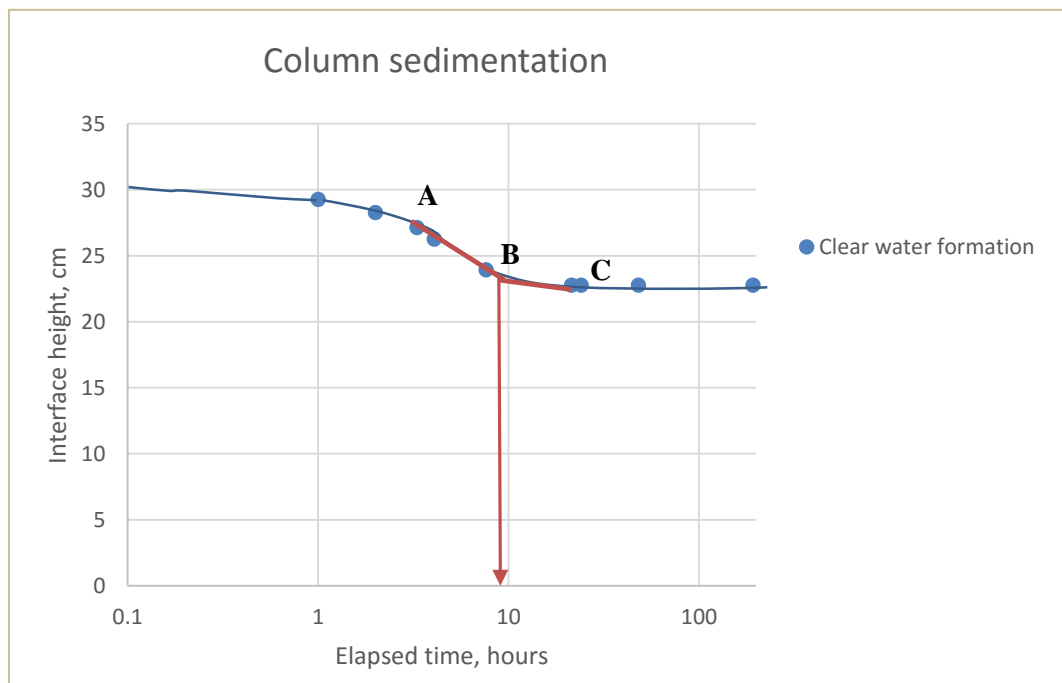


Figure 3-2 Sedimentation or self-consolidation process happening as the tailings sediments and the clear water forms. Interface height is the height of soil-water mix.

### 3.3.1 Rheological and Visco-Elastic-Plastic Properties

It was noticed that the slurry behaved as a viscous liquid during the first hour. Fastest change occurred in interface height, which then transferred to a much thicker, paste like, material in which excess pore pressure exists. With time, EPP dissipated and effective stresses increased over the tailings deposit's depth. The required time for self-weight consolidation was 457 min (7.6 hours), which is almost 7 times higher than the results of Rozalina and Yanful (2011). This is attributed the properties of the tailings such as the fineness of particles and the amount of fines, which affect the drainage properties, in the tailings under comparison are dissimilar. However, the same trend was observed in terms of the line (A-B) that defines the primary consolidation and the slightly descending line (B-C) thereafter that constitutes the secondary consolidation phase.

The test results show that if a layer of 30 cm maximum (height considered in the test) is placed on impervious layer, then the time required for contact stresses between soil particles would be around one day. It was estimated from interface level change data that the coefficient of consolidation of soil was  $0.5478 \text{ cm}^2/\text{min}$ . which lies close to the boundary established later by the Oedometric results. This is taken further and used for Oedometer test as will be discussed later in this chapter.

It is intuitive that as the slurry thickness increases, the time for the soil particles to sediment and for the effective stresses to form will increase consequently. However, the literature contains plenty of information that slurry thickness can be smaller as the reported heightening rates are in the range of 3 to 5 meters per year which translates to 42cm per month maxima. However, some mining companies are reported to use a quite shorter period of time for constructing layering that are within the aforementioned yearly limits, 10days is reported to construct 3meters layers

(Zardari, 2013). This makes assessing the excess pore pressure development of utmost importance for stability reasons. Furthermore, the sedimentation process is affected by the particle arrangements, angularity, specific gravity, size, and electrochemical repulsive/attractive forces. Therefore, caution should be exerted as to not rely on this estimate and map it to other tailings. Indeed, every tailings is unique in itself; therefore, its parameters, especially of importance to the project, should be investigated in isolation and then integrated together to get a full picture of the expected field behavior.

### 3.4 Characterization Tests

#### 3.4.1 Sieve Analysis and Hydrometer Tests

The sieve and hydrometer analyses are employed to establish the gradation of the tailings<sup>1</sup> material used in this study. Figure 3-3 presents the results of the tests conducted of oven-dried tailings<sup>1</sup> deposit in accordance with ASTM specifications (ASTM D6913, D422). The results are interpreted using the standardized ASTM procedure and Unified Soil Classification System (USCS) (ASTM 2487). Based on these results, the soil can be classified as a silt. Based on the obtained gradation results, the mean particle size,  $D_{50\%}$ , effective particle size  $D_{10\%}$ , Coefficient of uniformity,  $C_u$ , and Coefficient of curvature,  $C_c$  here determined and the results are presented in Table 3-2.



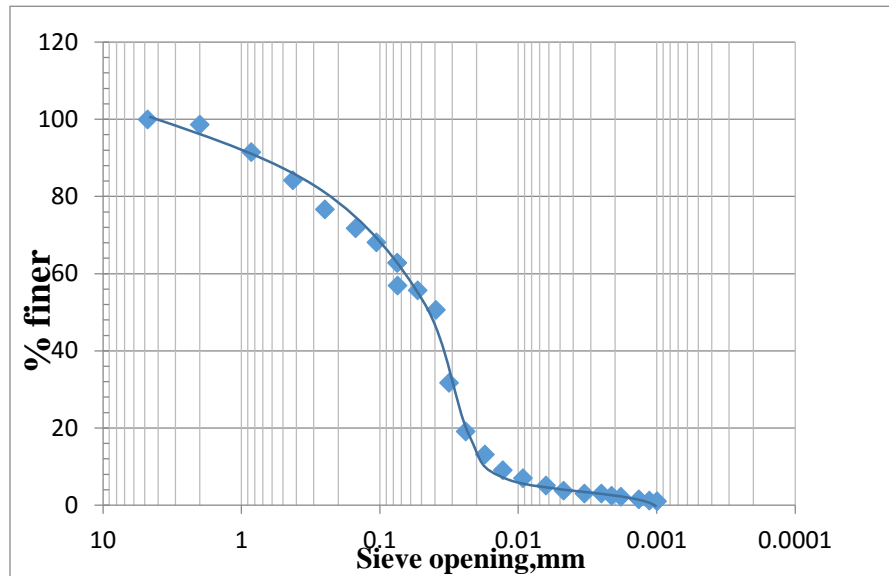


Figure 3-3 Sieve analysis test and hydrometer test's graph for tailings1.

### 3.4.2 Atterberg's Limit Tests

The oven-dried tailings1 was sieved through #40 U.S sieve, crushed with a pedestal for 10 minutes. The crushing effort was gentle enough to ensure not to destroy soil fabrics. The specimen was then tested to determine its liquid limit (LL) and plastic limit (PL) and plasticity index (PI) using Casagrande's apparatus in compliance with ASTM standard (ASTM D4318). The result of these tests revealed that the soil's LL is 48% and that it is non-plastic as it was not possible to roll it down to the specified diameter of 3.18 mm. in accordance with the standardized procedure in ASTM without crumbling. The results are also listed in Table 3-2.

### 3.4.3 Classification

According to the previous results, and by using ASTM D2487 (i.e. the standard for Unified Soil Classification System, USCS), the tailings are classified as low plasticity silt, ML.

### 3.4.4 Specific Gravity

The oven-dried tailings<sup>1</sup> was tested to define its specific gravity, commonly denoted  $G_s$ , in compliance to the standard method (ASTM D 854). The specific gravity was found to be 2.69.

***Table 3-2 General physical properties of tailings<sup>1</sup>.***

Index parameters	Value
USCS of the tailings	ML
Mean particle size, $D_{50}(\text{mm})$	0.045
Effective particle size, $D_{10}(\text{mm})$	0.0175
Coefficient of uniformity, $C_u$	3.4
Coefficient of curvature, $C_c$	0.8
Liquid limit, LL (%)	48
Plastic limit, PL (%)	NP
Liquidity index, LI	1.34
Specific gravity, $G_s$	2.69

## 3.5 Physical Properties of Used Additives

Two types of additives were used in this study: traditional and non-traditional admixtures. Traditional additives are the admixtures that are commonly used in practice, which have been known for long time (i.e. greater than 100 years) such as lime, cement, fly ash, gypsum, cement

kiln dust and slag furnace. The non-traditional additives are the type of additives that have been used quite recently such as polymeric derivatives, fibers, geotextiles, geogrids, and emulsified polymers. The aim of using these two types of binders was to improve the strength and stiffness of the tailings.

The general goal of this thesis is to improve the tailings that are used for the construction of an upstream tailings impoundments since the literature contains plenty of evidences they are the most likely to fail and the most reported to have suffered from major incidents (Davis et al, 2002). Such failure incidents were mostly attributed to the weakness of such soils due to their nature of disposal. Therefore, some additives are used in the laboratory study to evaluate their strengthening effects on the tailings, which will be evaluated through change in shear strength. These values will be later used in a numerical model to evaluate the effectiveness of these additives in improving the stability of the tailings dam.

### 3.5.1 Non-Traditional Additives

The non-traditional additive was shipped from U.S factory of a major Canadian leading industrial company, which requested confidentiality. As far as published research of polymeric emulsion is concerned, they are reported to have produced promising results. For instance, water soluble polymer was used in concentration of 20% in expansive soil to study its effects in reducing the erosion potential upon rain event and in the expansive potential (Huang and Liu, 2012). The results were promising in terms of the emulsified polymer improvement to erosion resistance. Newman and Tingle (2004) has reported that the polymer is usually emulated in potable water with 40-45% concentration along with 1-2% emulsifier. Examples of polymer emulsions used to treat tailings include;

- Vinyl acetate (Newman and Tingle, 2004).
- Acrylic-based copolymer (Newman and Tingle, 2004).
- Lignosulfonate with polyvinyl alcohol (Huang and Liu, 2012).
- Polycarboxylated acrylic acid polymer (Moghaddam, 2010).
- Polyacrylamide (Huang and Liu, 2012).

The used polymer in this study is 53% water and 47% solids. Its properties are listed in Table 3-3 and the physical properties of the resulting final product of emulsified polymer are listed in Table 3-4. The used emulsified polymer has the benefit of not only increasing the bonds between soil particles but also has a low viscosity that makes it easily permeating through the soil particles therefore enabling best mixing condition. It should be noted that it was reported that when using emulsified polymers, strength gain after one day of mixing is usually faster when the binder is polymer than when it is cement (Newman and Tingle, 2012). This is in agreement with manufacturer's statement that it assumes its full strength only one day after mixing which is a merit that this polymer has especially when time comes into equation.

***Table 3-3 Physical properties of the used acrylic polymer.***

	Resin	Part B	Accelerator	Hardener
Appearance	Clear liquid	White liquid	Clear liquid	White solid
Viscosity	5 cP (5mPa · s)	12 cP (12 mPa · s)	2 cP (2 mPa · s)	
Density	8.76 lb/gal (1.05 kg/L)	8.43 lb/gal (1.01 kg/L)	7.76 lb/gal (0.93 kg/L)	approx. 21.7 lb/gal (2.6 kg/L)
Notes*	*@ 68 °F (20 °C)			

***Table 3-4 Physical properties of the resulting final product of emulsified polymer.***

Index parameters	Value
Physical state	Liquid
Color	Transparent
Solvability in water	Solution
Viscosity (mPa.s)	13 (very low)
Sparking point	600
Density (kg/l)	1.07(20°)
Gel time (t)	0:48- 8:46 depending on the accelerator
PH	Not measured

### 3.5.2 Traditional Additives

The aim of using the traditional additive is to comparatively evaluate their effectiveness relative to the non-traditional additives and to optimize the best performing one. Therefore, recycled gypsum, B, mixed with CKD in different ratios was mixed in different percentages with tailings<sub>2</sub> received from Golder Associates. As stated earlier, tailings<sub>2</sub> was only used for the optimization work to establish the best functioning proportion of recycled gypsum and cement kiln dust. The percentages of the additives were 5, 10, and 20% of the total weight of the tailings samples. The recycled gypsum, B, stacks were generously provided by Rona whereas the CKD was provided by Lafarge. Batches were casted in plastic molds and tested for UCS at 7 and 14 days. The objectives of such study are two folds;

1. To investigate the effects of increasing recycled gypsum percentages in the mix proportion.
2. To reduce the cost of additives as the recycled materials are generally cheaper than the new produced materials.

Once the effects of gypsum are certified and the effects of increased additive's percentages are investigated, the best functioning proportion of B: CKD will be chosen and implemented with the previously tested tailings.

## 3.6 Testing

### 3.6.1 Oedometer Test

The tailings were received in a slurry form, that is to say they were inundated with water in excess of their liquid limit (LL), and LI was 1.34. Therefore, their characteristics were difficult to establish without knowing how they will transform from liquid to soil sediments and how long

that will take. This was achieved using the Sedimentation Column. Next, the tailings void ratio at specific depth, or overburden, was determined from Oedometer test results, where the soil slurry was molded in a bottom-perforated cylindrical mold and then allowed to self-settle for a day and then pressurized with 1.25kPa for 24hours to reduce its water content and to establish grain-contact between soil particles. Figure 3-4 shows the setup used for dewatering the slurry to zero-effective stress void ratio. The setup comprises a plastic cylinder, a porous stone and a filter paper.

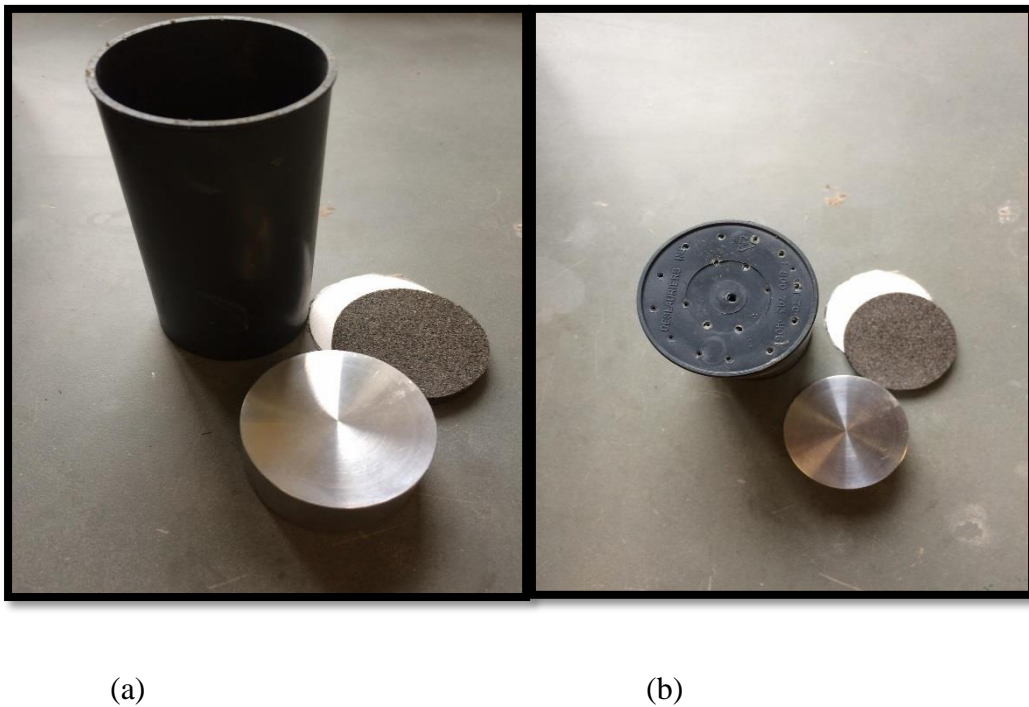


Figure 3-4 a) the tools used for the dewatering setup; b) perforation to allow collecting water from bottom.

The boiled porous stone and filter paper were used to seal the bottom to ensure that water goes out from the perforations when the viscous tailings are squeezed. The load was applied for 24 h based on the soil behaviour observed during the sedimentation column where that excess pore

water pressure dissipated and soil consolidated within one day. After 24 h of dewatering, the soil was transferred to the Oedometer ring, with measuring  $W_c\%$ , and tested in the oedometer according to the ASTM standard (ASTM D2435). The sample was allowed to consolidate in the oedometer for 12 h. The initial water content was slightly higher than LL, 51.1%, which demonstrates the effectiveness of the dewatering technique used before starting the consolidation test. The coefficient of consolidation,  $C_v$ , measurements were taken for the untreated tailings at different loading steps to establish lower and upper bounds. Figure 3-5 shows the  $e$ - $\log \sigma'$  curve for the untreated soils.

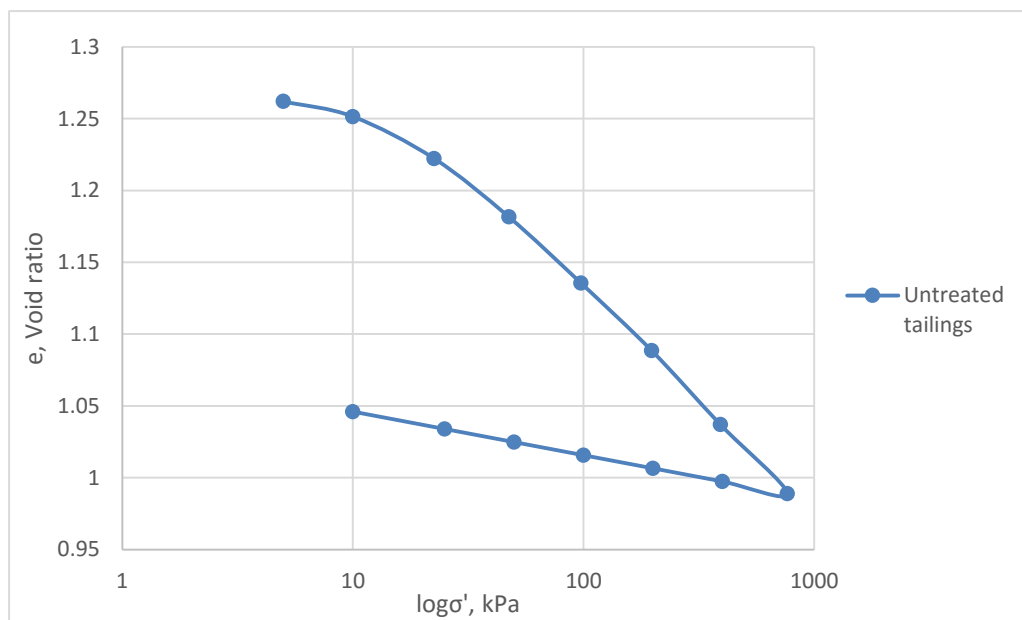


Figure 3-5  $e$ - $\log \sigma'$  curve of the untreated tailings.

The coefficient of compressibility indicates the potential consolidation settlement upon loading cohesive soils. The typical values for  $C_c$  are in the range of 0.1-0.2 for silty soils are 0.2-0.3 clayey soils (Vick, 1999). It is estimated by the slope of the best fit straight line of virgin consolidation curve, i.e.



$$Cc = \frac{e_1 - e_2}{\log\left(\frac{\sigma'_2}{\sigma'_1}\right)} \dots\dots\dots (3-1)$$

Where  $e$  is the void ratio and  $\sigma'$  is the effective stress. The two subscripts 1&2 are any two arbitrary points on the virgin consolidation line.

No rebound measurements for the untreated materials were recorded. Therefore, only  $C_s$  values were compared to treated tailings. Based on equation 3-1,  $C_c$  was estimated to be 0.1548. Figure 3-6 presents the volumetric strain of untreated tailings versus  $\log \sigma'$ , which indicates the amount of consolidation settlement. It should be noted that one of the desired outcomes of treating the tailings is to reduce the coefficient of compressibility of the treated tailings to minimize the consolidation settlement.

The  $C_v$  values were established and plot with  $e$ - $\log \sigma'$  and a general increase of their value was observed as effective stresses increase. This is attributed to the tailings becoming stiffer as the effective stresses increase and therefore draining quicker at higher stresses. Figure 3-7 shows the values of  $m_v$  as well where it is clearly observed that stiffness increases with effective stresses increase. The coefficient of consolidation,  $C_v$ , ranged from 0.1 to 0.5 cm<sup>2</sup>/minute as shown in Figure 3-7.  $C_v$  measurements were done using Taylor (1942) method (Das, 2006).

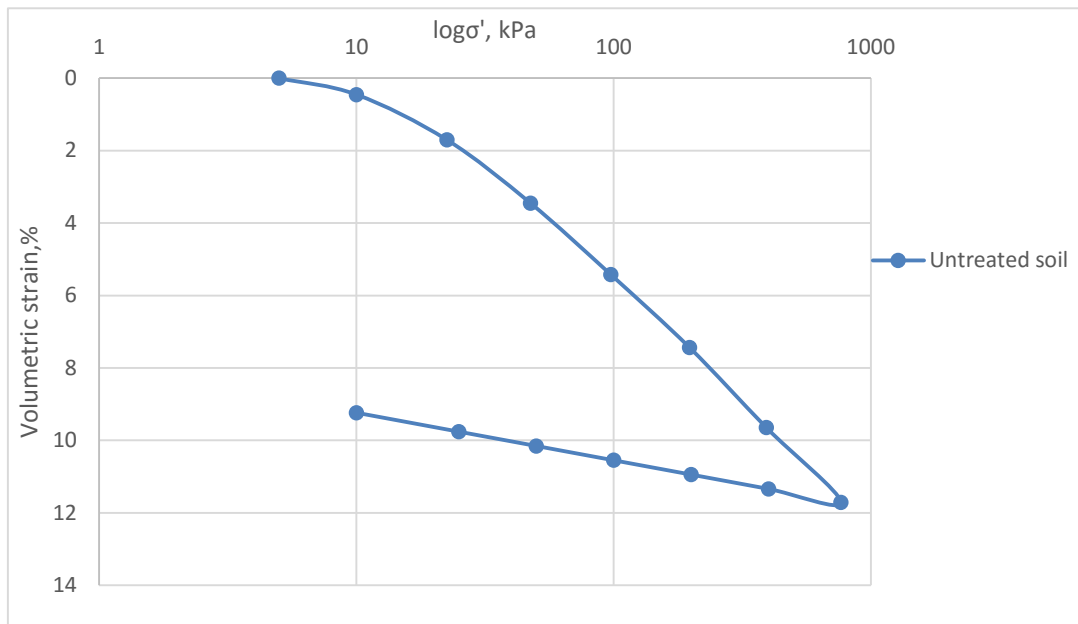


Figure 3-6 Volumetric strain against effective stress of untreated tailings

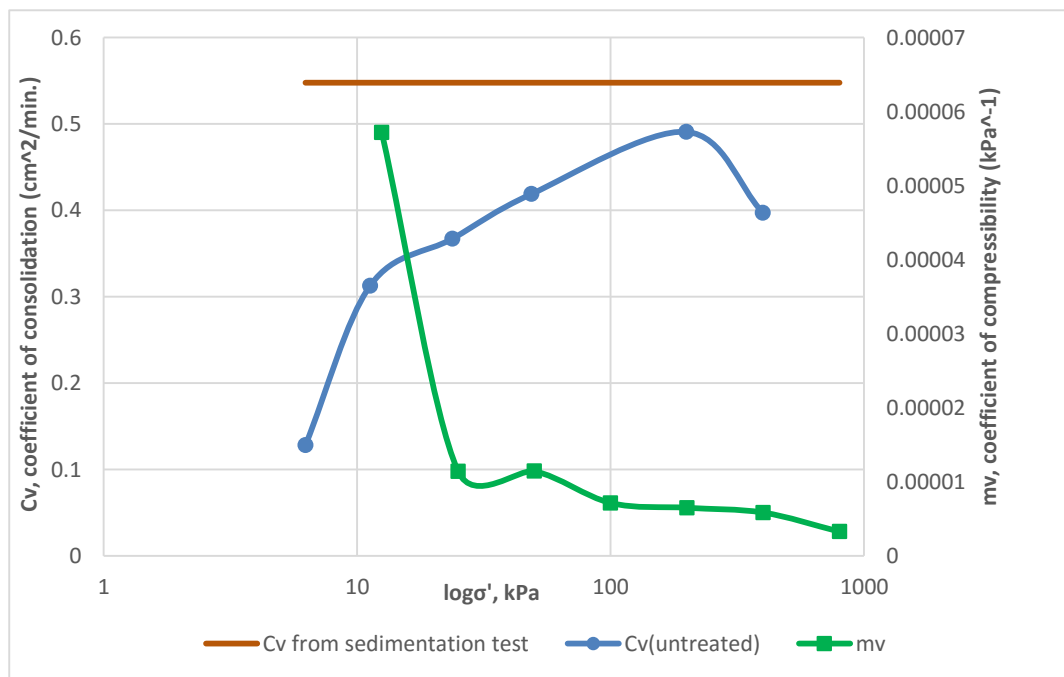


Figure 3-7  $C_v$  and  $m_v$  values at different effective stresses of untreated tailings.

It is seen from Figure 3-7 that  $C_v$  obtained from the sedimentation column is much higher than the average  $C_v$  value of  $0.38 \text{ cm}^2/\text{min}$ . It is also noticed that the tailings exhibited a volumetric

reduction that amounted to 17.05% of the original volume when the tailings was consolidated to 765kPa.

### 3.6.2 Shear Strength

One of the main objectives of this thesis is to assess the strength parameters of the tailings before and after the treatment. The strength parameters of untreated tailings provide the base line for the shear strength with regard to which the shear strength of treated tailings will be compared. To evaluate the shear strength parameters, two main laboratory tests may be conducted: the direct shear test (DST) and triaxial test (TT). These tests allow measuring the strength parameters in accordance with the anticipated soil behavior, whether drained or undrained and either consolidated or unconsolidated. In this regard, the tailings behavior expected in the field is neither fully drained nor undrained because of the unpredictable soil gradation a specific distance away from the discharge point. However, the analysis considered in this thesis is effective stress analysis, ESA, in which the resulting EPP is taken into account in the numerical model along the potential slip surface length. Therefore, it was decided to use drained DST to determine the strength parameters of the treated and untreated tailings. Results are reported in section 3.7.

#### 3.6.2.1 *Direct Shear Test (DST)*

DST is a test conducted to establish the shear strength parameters of a soil by measuring its resistance to shearing at a predetermined plane. This is achieved by placing the soil in a cell split in two halves at its mid-height sandwiched between some platen arrangements. The plates can be solid or perforated depending on the aim of the test. If drained test is considered, then perforated plates are used with filter papers and porous stones to allow for drainage. Further, one or two way drainage can be specified in the test depending on the problem in hand. The device used in

this testing has a square box 60x60 mm with fully computerized data logging system. It has two LVDTs, one for vertical and the other for horizontal displacements, which are hooked to the Daisy lab logging program. The accuracy of LVDTs is  $\pm 0.01$  mm. The machine is equipped with two normal loading cells, one vertical to the box and the other is shifted to make use of the mechanical loading using a lever arm. The maximum normal pressure that can be applied is 0.6 MPa.

#### *3.6.2.1.1 Sample preparation and test criteria*

The test is conducted in accordance with the standardized ASTM procedure (ASTM D 3080). The DST was conducted using a strain controlled machine with a rate of 0.03 mm/min with stroke limit of 16mm. The strain rate was chosen to ensure that EPP does not occur due to shearing. It should be noted that there is no rule of thumb to decide on the rate of shearing and it might take some trials to establish the suitable rate especially for water soaked samples. At least three samples of each material were prepared and tested at 3 different normal stresses to establish the shear strength envelope of the material tested. For untreated tailings, the specimens were prepared using the oven-dried tailings at void ratios that correspond to those established using the Oedometric results. A square shear box of 60mm dimensions was used for this purpose. As seen from Table3-3 and Figure3-3, the mean particle size of tailings was 0.045mm and the maximum was 2mm. The ratio of the inner dimension of the shear box to the largest particle size encountered was 30, which is higher than the minimum established ratio of 10 in literature and laboratory standards. Further, the thickness to maximum particle size was 20 which is also higher than the minimal value of 6 (Benson et al., 2008; ASTM D 3080).

### 3.6.3 Tailings Treatment

#### *3.6.3.1 First Approach to Soil Improvement*

The tailings tested in this thesis were two types. First one was silty tailings denoted by tailings1 in Table 3-1. The second one was clayey one and was denoted tailings2. All testing was done on tailings1 except UCS, optimization work, which was done using tailings2. Therefore, to eliminate any confusion, all results except for UCS are for tailings1. It was decided to evaluate the soil improvement with polymeric emulsions employing Oedometer testing. The polymeric emulsion used in this study is a rubber like material when it hardens. The supplier of this material stated that it is quick hardening material that attains its full strength in 1 day. It should be noted that the unconfined compression strength (UCS) of this material was 1MPa. Also, Oedometer testing provides a wide picture of soil behavior and very useful results required in the analysis, including: hydraulic conductivity, constrained modulus of elasticity and coefficient of consolidation. These results are essential parameters for Mohr-Coulomb model.

##### *3.6.3.1.1 Sample preparation*

The tailings were received in a slurry form as previously stated. Its treatment was accomplished by adding polymeric emulsion material to provide bonding to the tailings particles. Three percentages were investigated in this regard. The polymer was prepared in accordance with the supplier's product instructions for preparation and mixing. It was prepared by mixing the liquid polymer with a specified amount of hardener and accelerator shortly before being mixed with the slurry. Afterwards, the final product of polymer was poured with the slurry into a mixer and mixed for 5minutes. The final product's hardening time commences eight minutes after its two main components are proportioned and mixed. Therefore, it was crucial to ensure that the tailings were first poured into the kneader's bowl before the two components are mixed

to avoid hardening. Next, the resulting batches were casted into a cylindrical molds 100 mm height and 50 mm in diameter. Since the polymer was not expected to have hydrophilicity, the batch was allowed to self-settle by providing bottom drainage. The plastic molds used were perforated from the bottom and sealed with porous stone and a filter paper to retain the soil particles. Once poured into the mold, a seating pressure of 1.25kPa was applied to the batch to help initiate zero effective stress void ratio and allow soil-soil contact to happen. The seating load was applied by an Aluminum ring which was lubricated around its edges to minimize friction. The polymer role will be bridging the gap between those particles and reducing the tailings' water content. The investigated percentages were 0.5, 1 and 2% by total weight..

### *3.6.3.1.2 Oedometer results*

After 24 h from casting the treated material as described above, the tailings were demolded into the consolidation rings. Care was exercised as to not disturb the tailings by remixing, little shaking, without taping, was followed to allow the batch to take the shape of the Oedometer's ring. Once filled, the surface was leveled to ensure clean surface and to avoid cap tilting. Load cap was then applied and the Oedometer box was then transferred to loading frame. The specimens were then loaded according to the established ASTM standard procedure and following the procedure mentioned in section 3.5.1. Figure 3-8 shows the obtained  $e$ -log  $\sigma'$  curves for the tested material. It is noted from Figure 3-8 that adding polymers to the soil matrix increased the solids content to the tailings which resulted in a reduced initial void ratio, which is attributed to dewatering-like mechanism. It resembles dewatering, though slurry's water content was not reduced, in that when solids are added, the water content will decrease. This is supposed to increase the dry unit weight and therefore the strength and stiffness parameters.

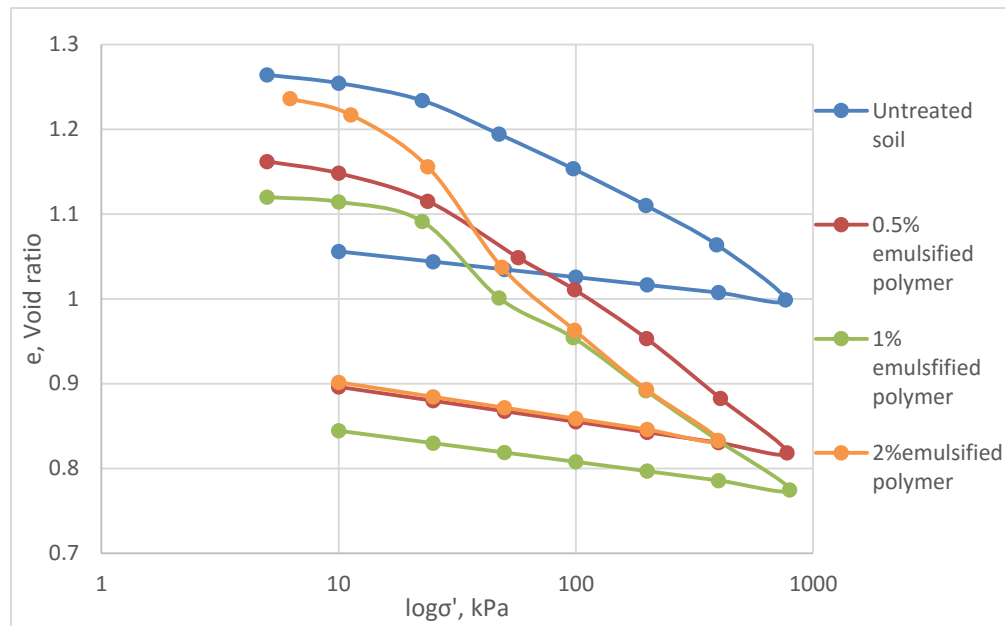


Figure 3-8 Void ratio- logarithmic effective stress plot of the different investigated percentages.

Figure 3-9 shows the volumetric strain versus logarithmic effective stresses relationship. It should be noted that the strains recorded before the applied pressure of 5kPa were removed for all three curves to better capture the real behaviour of the polymer-improved soil at high stresses. It is seen from Figures 3-8 and 3-9 that the addition of polymer did not stiffen the tailings, though higher dry unit weight was achieved. This is attributed to the fact that the polymer used was rubber-like material which is more compressible than soil particles.

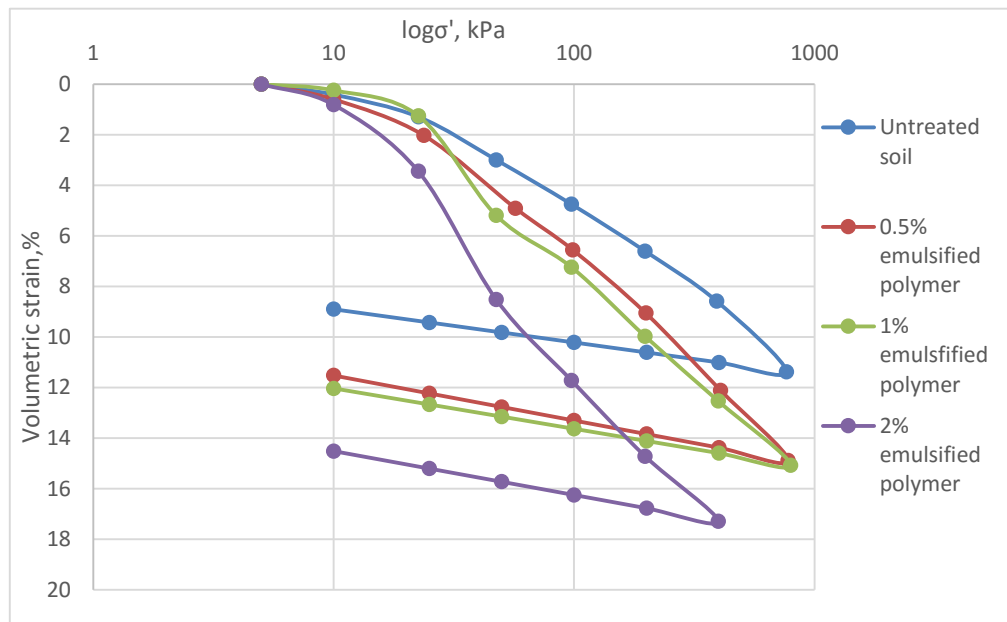


Figure 3-9 Volumetric strain-logarithmic effective stress relationship.

The desired improvement is stiffer soil, which should be manifested in lower values of compression index,  $C_c$ . For untreated soil, the compression index value was 0.1548, while for tailings treated with 0.5, 1 and 2% polymer,  $C_c$  was 0.2265, 0.1928, and 0.2302, respectively. This clearly shows that the untreated tailings were stiffer than those treated. It is unclear though why the 1% was stiffer than the 0.5% and 2%. Nevertheless, for the three percentages it was clear that the polymer increased the compressibility of the tailings. It should be mentioned that these percentages were chosen because they are reported to have been used in literature and also for economic purposes (Naeini et al., 2011; Arioglu et al., 1985; Lightsey, 1969). Moreover, these small percentages are large quantities since the liquid polymer has a unit weight as that of water which, in turn, results in large volume being added to the tailings. To sum up, adding the commercial type of emulsified polymer resulted in softer tailings based on Oedometric results. It was observed that the compressibility and consequently the volumetric strain were increased



when using this type of polymer. Therefore, the treatment with this polymer was not pursued any further in the current study.

### *3.6.3.2 Second Approach for Soil Improvement*

As the polymer treatment results were not promising, the author considered the traditional additives to achieve the desired outcomes. Ahmed et al. (2012) reported that a mixture of recycled Gypsum, Bassanite (B), and cement kiln dust (CKD) resulted in substantial improvement to tailings. They also reported employing recycled gypsum to improve waste disposal location for reclamation purposes. Therefore, B, and CKD were considered as candidates to be used for tailings treatment in the current study.

Tailings<sub>2</sub> with properties as reported in Table 3-1 were used in this study. The results from this study can also be used as an indication for the viability of the applied treatment with the silty tailings (Tailings<sub>1</sub>). In particular, the treatment scheme used herein is tailored to give an indication on the expected performance of CKD: B admixture in strengthening the tailings. Unconfined compression strength testing was employed to determine the strength of the soil treated with the CKD: B admixture and other relevant proportions. Five combinations of CKD: B were prepared at 5, 10 and 20% by the total weight of the tailings and were subsequently mixed with the tailings<sub>2</sub> utilizing a kitchen kneader. All test results are reported together for each batch to provide an opportunity for comprehensive evaluation of the treated soil behaviour.

#### *3.6.3.2.1 Unconfined compressive strength (UCS) tests*

The tests were conducted in accordance with ASTM standardized procedure (ASTM D 2166). The CKD: B mixture was prepared in three different percentages: 5, 10 and 20% to the total weight of the tailings<sub>2</sub> used in this research. The recycled gypsum was milled and sieved through #20 U.S sieve and heated for at least 2 hours in oven at temperature of 140°C. Figure 3-10 shows

the recycled gypsum and the final product, Bassanite. Similarly, the CKD was milled and sieved through #40 US sieve. The recycled gypsum and CKD were then mixed together. The mixture was then proportioned according to Table 3-5 and dry mixed before slowly being added to the slurry while being stirred. The slurry was mixed with hand mixer thoroughly to ensure homogeneity.



Figure 3-10 Preparation of Bassanite, a) recycled gypsum (B) ready for grinding; b) final product.

**Table 3-5 Combination made for optimization work for traditional admixture.**

Percentage of CKD: B	Ratio B:CKD	Combination
5%	1:1	5- 1
	1:1	10- 1
10%	2:1	10- 2
	4:1	10- 3
20%	1:1	20- 1

The slurry was weighed and placed inside a kitchen kneader and was stirred after adding the admixture thoroughly for 5 minutes, where the rotation speed was increased gradually to further ensure homogeneous samples. The resulting batches were casted in three lifts into greased cylindrical molds 50 mm in diameter and 100 mm high. Each lift was tapped 10 times with a 2mm diameter rod, ensuring the tapping is distributed evenly across the area of each lift (Madhyannapu et al., 2010). Finally, the cylindrical molds were placed on the shaking table for 120 seconds, which is shown in Figure 3-11.



Figure 3-11 Shaking table used to expulse air within.

The casted samples were exposed to air at normal room's temperature for 24 hours. Afterwards, specimens were capped with lids till they were tested after 7 and 14 days curing. UCS tests were carried out using a strain controlled machine with strain rate of 4.3mm/min. The machine was calibrated for both the range of applied loads and the strain rate before testing to ensure reliable results.

Typically, most of the strength is achieved in the first 28 days when traditional admixtures are used (Bertero and Valagussa, 2012). On the other hand, soil treated using gypsum achieves most of its maximum strength in the first 14 days (Ahmed et al., 2012). Therefore, to evaluate the strengthening effect of the soil treatment, the treated soil was tested after 7 days and after 14 days. In total, thirty samples were casted for the five combinations, half of them were tested after 7 day of curing and the other half was tested after 14 days. Thus, for every combination, 3 samples were tested after 7 days and 3 samples were tested after 14 days. After the specified curing time, the samples were placed in the UCS test machine as shown in Figure 3-12.



Figure 3-12 UCS Sample setup. The device is equipped with a dial gauge to measure the vertical displacement.

During the test, the shear was accomplished over duration of 240 seconds. Figure 3-13 shows a typical sample after the completion of testing. As can be noted from Figure 3-13, the failure plane was not well-defined for sheared samples. After shearing was completed, the water content was measured to establish the dry unit weight.

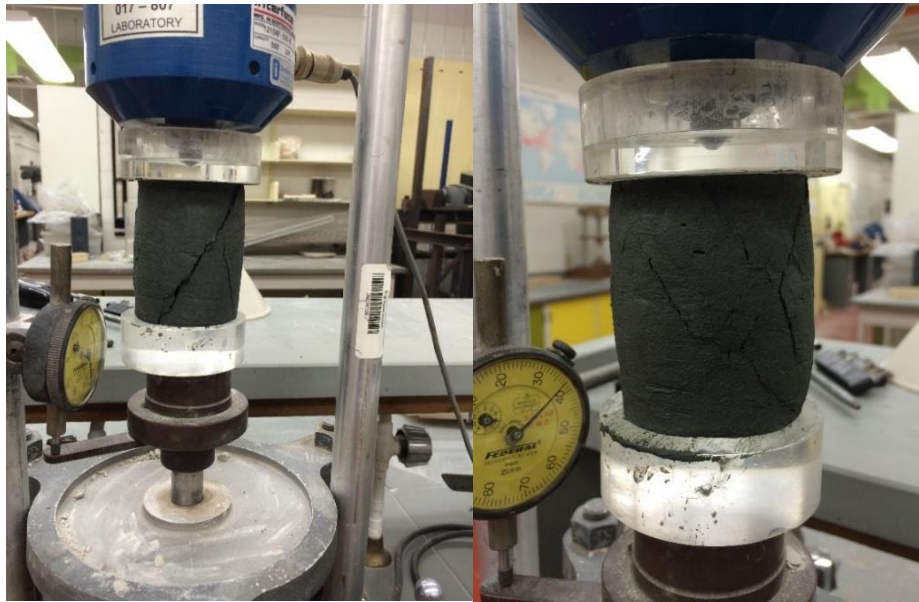


Figure 3-13 Failure plans for two sheared samples.

### 3.6.3.2.2 *Effects of internal forces on the treated soil*

Under undrained conditions, the maximum undrained strength is not a function of principal stresses,  $\sigma_1$  and  $\sigma_3$  and therefore, theoretically speaking, applying confinement without allowing consolidation does not affect the undrained strength. However, for applications where vertical stresses and associated settlement will vary with time, it is important to recognize the changes that occur to the treated material and how these variables will contribute to the treated material's parameters.

Wetting and drying cycles can also induce volumetric changes to the treated soil whereby altering its internal structure. This situation should be taken to consideration. Should the treated soil lies within freeze and thaw active region, its bonds will inevitably be subjected to internal freezing and thawing forces which could ultimately jeopardize their integrity. Therefore, it is important to address these issues early in the design stage to avoid un-favorable situations where the field parameters diverge from what they should be. In this thesis, no freeze and thaw tests were done to assess their influence on the structure of the soil; however, consolidation tests were conducted to a maximum stress of 800kPa (typical range of mine tailings embankments is between 0-2MPa) in order to assess whether or not a certain stress or an accumulation of stresses could induce bond's breakage and that will be discussed in the following sections.

### *3.6.3.2.3 Effects of curing days*

Figure 3-14 presents the variation of unconfined compressive strength of the tailings treated using the different schemes with time. It is noted from Figure 3-14 that decreasing the proportion of the CKD resulted in a decrease in the strength of the treated soil. This is because as the gypsum proportion increased, the CKD proportion decreased and CKD has greater strengthening effect than the recycled gypsum. On the other hand, increasing the percentage of the admixture in the tailings resulted in an increase in the strength. It should be noted that the recycled gypsum was sieved through #20 U.S sieve, which indicates coarse particles similar to fine sand. On the other hand, the CKD which was sieved through #40 U.S sieve size. Thus, the surface area of the gypsum is small compared to the CKD, which means reduced efficiency with regard to water absorbency. This means the outer surfaces of the gypsum particles absorb the water and set without their inner part necessarily absorbing any water. This allows the soil treated with gypsum to have higher  $W_c\%$  than the soil treated with CKD, or if the soil was



treated with finer gypsum. It is also noted from Figure 3-14 that the strength of the treated soil increased as the curing time increased from 7 days to 14 days. However, the percentage increase in strength was approximately 10%. This indicates that most of the improvement occurs within the first 7 days. This is advantageous for applying the treatment to tailings dams as it allows early gain in strength.

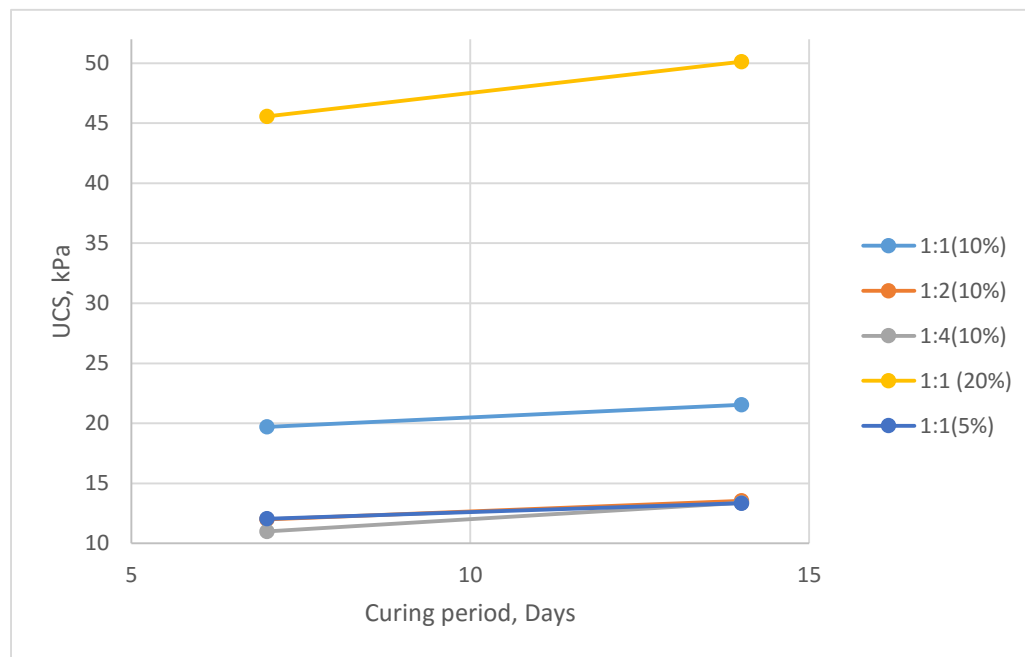


Figure 3-14 UCS of the five combinations for the period of testing.

#### 3.6.3.2.4 Effect of commercial gypsum's proportion of UCS

The results showed that the unconfined compression strength decreased as the proportion of the recycled gypsum in the mix increased, which was attributed to the relative higher effectiveness of CKD. However, it is worth exploring if recycled gypsum has an adverse impact on strength of the treated tailings. Therefore, powdered gypsum, also known as plaster, was used for the treatment of tailings instead of the recycled gypsum. The same aforementioned procedure for sample preparation and curing was used for the sample treated with combination of 10% admixture was followed. Nine samples were casted and allowed to cure for 7 days and then

tested. The results obtained from the tests on specimens including plaster (denoted with letter P) are compared in Figure 3-15 with the results obtained for the same combination but with recycled gypsum. It is noted from Figure 3-15 that the samples treated with admixtures including plaster exhibited higher UCS as the Plaster proportion increased. For example, UCS of treated tailings increased by more than 100% when the CKD: Plaster ratio was varied from 1:1 to 1:4.

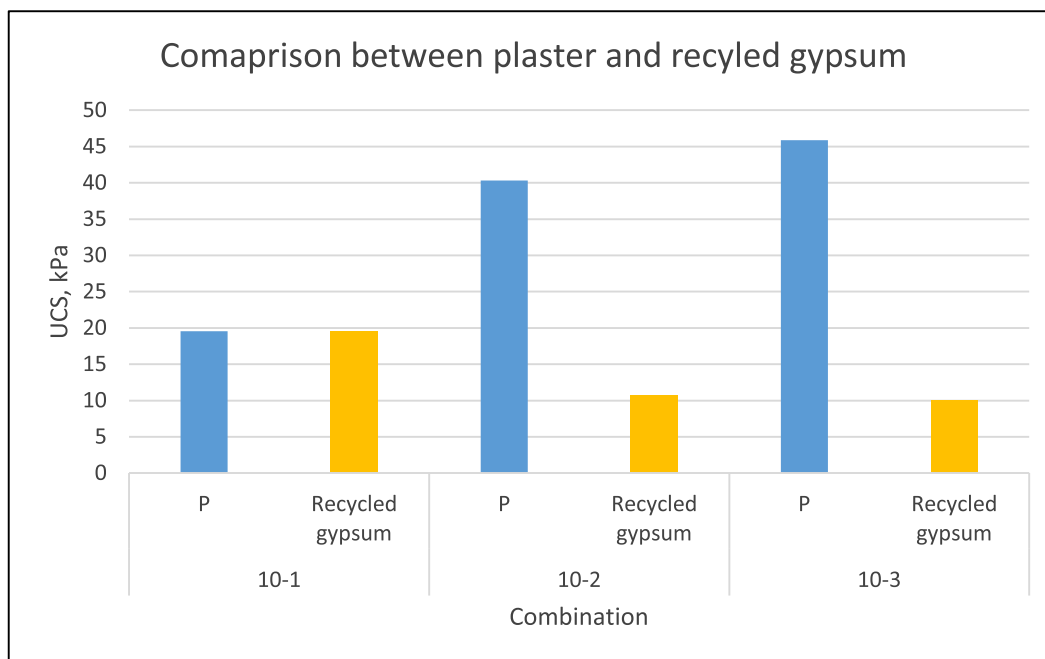


Figure 3-15 UCS of tailings treated with 10% admixture with different CKD: Gypsum (recycled and commercial) proportions.

These results clearly indicate that gypsum can be effective in tailings treatment. The inferior performance of recycled gypsum in tailings treatment can then be attributed to the following two reasons: the small surface area of the recycled gypsum, which in turn affected the hydrophilicity gypsum particles; and the lower amount of calcium oxide, CaO, of recycled gypsum (compared to plaster) as can be noted from XRF analyses' results shown in Table 4-1.



The stress-strain behaviour of treated tailings is important for accurate modeling of their performance as a construction material for the tailings dam. Figure 3-16 presents the stress-strain curve of specimen treated with 10% admixture (10-2). All other specimens displayed similar stress-strain behaviour. All specimens showed some strain softening with shearing; however, no peak-post peak behaviour was observed.

It was noticed that the proportion of gypsum had no clear effect on the cracking strain as all samples developed cracks at approximately the same strain level. However, the inclusion of cementitious material has changed the typical  $45^\circ$  cracking plane to multiple planes of cracking.

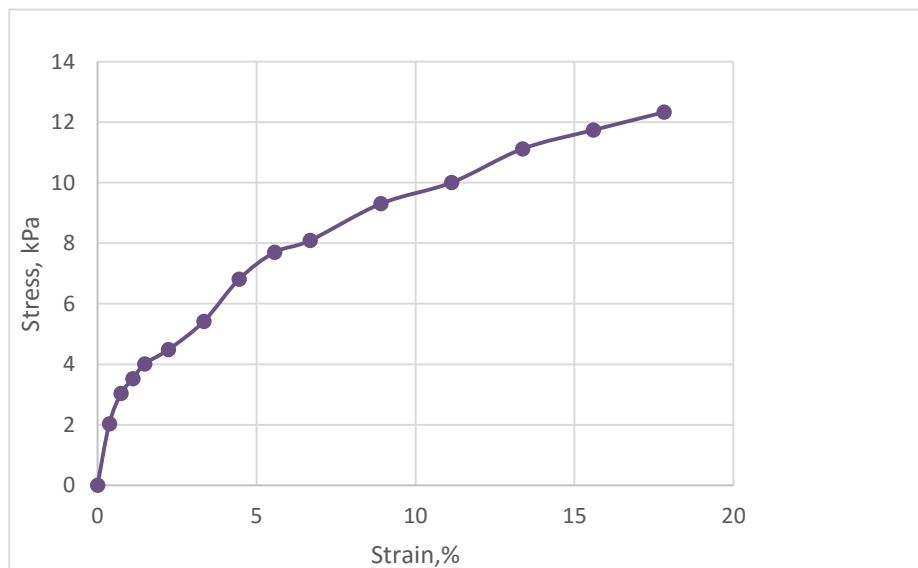


Figure 3-16 Stress-strain relationship of tailings2 treated with (1CKD:2B) at 10% obtained from the UCS tests.

Based on its good performance in treatment of the clayey tailings, the admixture of CKD: B with 1:1 ratio was selected for consideration of treatment of silty tailings (tailings 1). Two different percentages were considered for treatment of tailings 1, and the treated tailings were tested to evaluate their consolidation behaviour employing oedometer tests, and to determine their strength parameters using direct shear tests.

### 3.7 Sample Preparation

Two different treatment schemes involving cement kiln dust (CKD), bassanite (B) and ordinary cement (OC) were applied to tailings 1. The treated tailings were prepared by mixing the tailings1 slurry with: 7.5% of admixture (1CKD:1 B: 1OC); and 7.5% of admixture (0.45CKD:0.45B:0.1OC). Ordinary cement was added to the mix because the DST samples were soaked in water one day before testing and it was observed that those samples without cement disperse almost immediately after being placed in water. Ahmed et al. (2012) attributed that behaviour to the fact that the calcium sulfate hemihydrate being soluble in water.

The slurry was first poured into the bowl of the kitchen kneader and the admixture was slowly poured in while stirring the slurry. The rotating speed of the kneader was increased gradually. The time of mixing was set to be 5 minutes to avoid bleeding or segregation of the mixture. The resulting batch was then poured into specially fabricated square boxes 62 x 62 x 20 mm. The batch was kept in the normal room temperature. Figure 3-17 shows freshly casted treated samples.



Figure 3-17 Freshly casted treated tailings1 DS samples.

### 3.7.1 Dispersion

Initially, two treatment schemes were applied to tailings 1: 10% by total weight of admixture (1CKD:1B); and 7.5 % by total weight of admixture (0.45CKD: 0.45B: 0.1OC), were casted and left to cure in room temperature for 3 days. Afterwards, they were soaked in water for saturation before testing. However, it was observed that the samples treated with 10% admixture of (1CKD:1B) dispersed in the water (i.e. lost their bond) shortly after being deposited in the water, which indicated weak bonds formation. On the other hand, the samples with 7.5% admixture of (0.45CKD:0.45B: 0.1OC) have maintained their shape. Therefore, a third treatment scheme was applied; another set of samples were prepared with 7.5% by total weight of the admixture (1CKD: 1B: 1OC) following the aforementioned procedure. However, the curing period was 7days. These new samples (with ordinary cement) maintained their shape (i.e. bonds) after being deposited in water.

After the specified curing time was completed, the treated samples were placed in the direct shear box and were tested at different normal stresses to determine their shear strength. The normal stresses were applied in increments starting from 10 kPa and doubling the normal stress in each increment until to the desired stress was achieved. Similarly, treated samples were placed in oedometer rings and were tested to determine their consolidation behaviour. All samples were dry cured in room temperature and were plastic wrapped to prevent desiccation formed by moisture migration.

### 3.7.2 Test Results

The results of the DS and Oedometer tests are presented and discussed in this section.

### 3.7.2.1 Direct Shear Test

It should be mentioned that for the treated materials tested in the DS, each load increment was kept constant for one hour, which is considered to be long enough for all excess pore water pressure to dissipate. This is based on the lowest coefficient of consolidation that was obtained from the oedometer tests to measure the time for full dissipation. The untreated tailings, however, were prepared from oven-dried tailings that were crushed and compacted to achieve void ratios, which correspond to the effective stresses from  $e$ -log  $\sigma'$  curve. Consequently. They were kept under pressure and soaked with tap water and left to saturate for 24 hours. Very slow loading was applied (strain rate of 0.03 mm/min) to ensure shear induced EPP is dissipated. It should be mentioned that the water used here is a tap water for both tests to avoid result's bias. The tailings are usually submerged in acidic environment and the effect of this acidity should be given consideration. It is reported that pH variation has an effect on the shear strength parameters of the soil (Ghobadi et al., 2014). Figure 3-18 shows one sample after it was sheared.

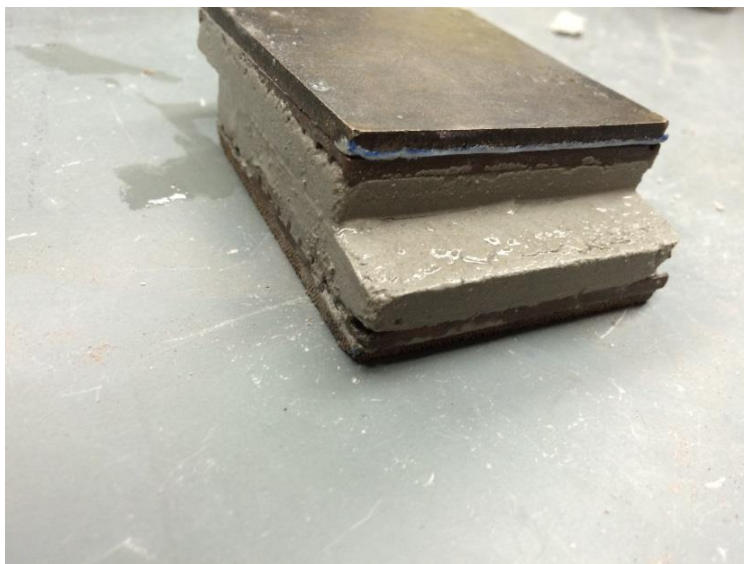


Figure 3-18 Untreated tailings Sheared DS sample.

Figure 3-18 shows that the failure plane was horizontal along the contact of the upper and lower parts of the untreated sample; however, there was some deviation from the horizontal failure plane for the treated soil, which was caused by the cementitious bonds that forced the failure to happen on the weakest plane, deviating slightly from the horizontal direction.

Figure 3-19 displays the shear strength envelop for the untreated tailings while Figure 3-20 displays the shear strength envelop for the treated tailings with 7.5% (1 CKD: 1B: 1OC). As can be noted from Figure 3-19, the shear stress-normal stress relationship was linear with a constant slope representing an angle of internal friction equal to  $42^\circ$ . However, the shear stress-normal stress relationship exhibited an intercept, which suggests the untreated tailings material has some cohesion. The observed apparent cohesion may be attributed to two possible reasons;

- 1- Effects of thixotropy: Janzak et al. (2007) and Rout et al. (2013) reported that the tailings exhibited some cohesion intercept. This also could be supported by the observation from  $e$ -log  $\sigma'$  curve (Figure 3-4) which exhibits an initially curved line at low stresses but became straight line at pressure higher than 25kPa. This observation, however, might be attributed to the fact that the sample was tested at water contents higher than its LL and the initial part of the curve does not necessarily have to be a straight line.
- 2- The  $c'$  intercept is a result of test's errors related to varying dry density and void ratio of tested specimens. Indeed, achieving a specific dry density at confining pressure and high void ratio is almost impossible to attain. The tailings were prepared at low dry densities, in the order of  $1200\text{--}1400\text{kg/m}^3$ , and they were soaked in water and loaded to reach the desired normal stress. This led to inevitable settlement, which in turn altered the initial void ratio and consequently the dry density which led to some over consolidation of the specimen that resulted in the apparent  $c'$  intercept. The author is of this opinion this is a

more plausible explanation. This is supported by the fact that the  $e$  values established after shearing the samples were all around 8% lower than the initial void ratio at which the samples were prepared.

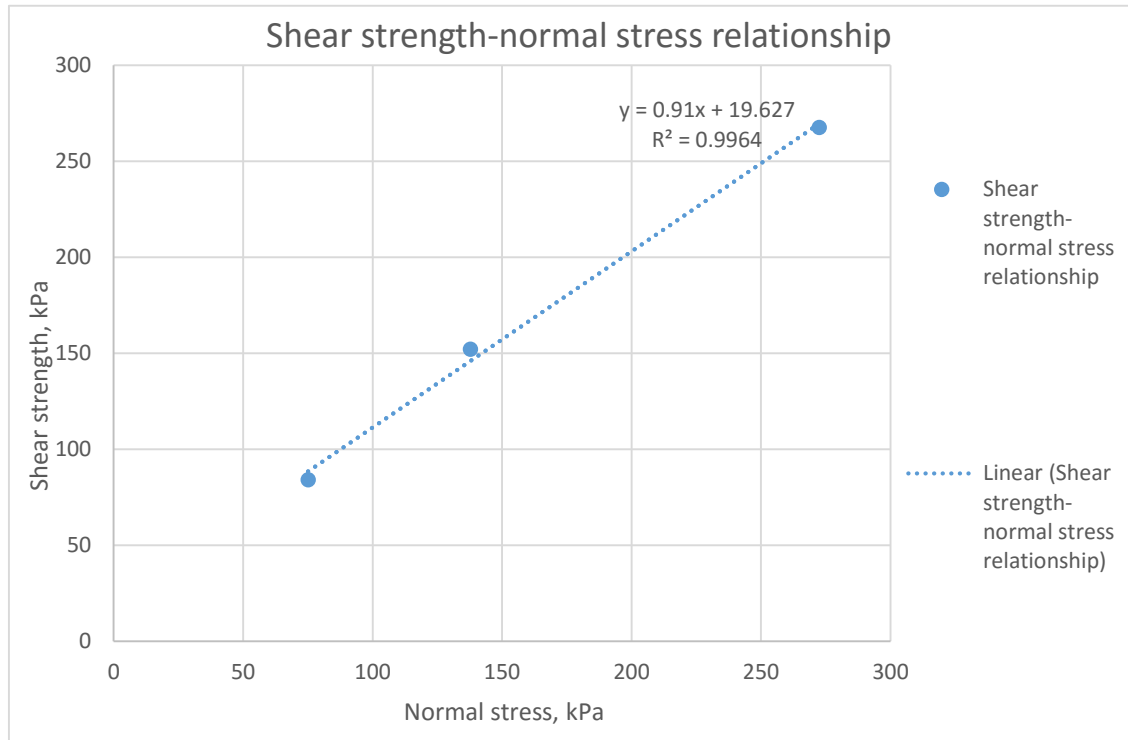


Figure 3-19 Shear strength envelope of untreated tailings.

Figure 3-19 shows that the peak shear stress ( $\tau_f$ ) - normal stress relationship for the untreated tailings is not linear, but rather is semi-exponential, which entails that the friction angle and cohesion of the treated tailings are not constants as typically assumed. The  $\tau_f$ -N best fit line might flatten at high stresses, which means that there could be a limiting effective stress beyond which the shear strength becomes constant. Zardari (2013) reported that Vick (1990) had a similar observation where he noted that the friction angle decreased until it stabilized at normal stresses higher than 276kPa (Zardari, 2013). This may be an important observation for assessment of tailings dams' stability, especially for high dams of more than a hundred meters, i.e., effective stresses that could reach a few MPas. If a constant friction angle is assumed in the

numerical modeling to assess the stability of such tailings dams, then it might be possible that the safety factor is overestimated.

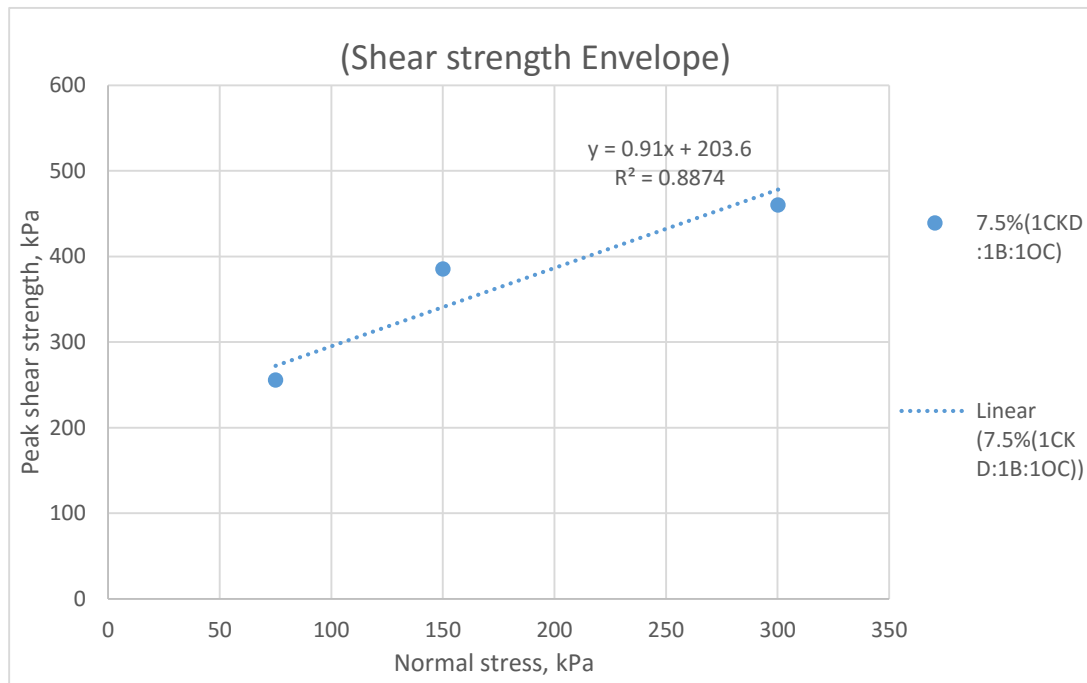


Figure 3-20 Shear strength envelope of 7.5 % (1CKD:1B:1OC) treated tailings.

Figure 3-21 compares the shear strength envelope of tailings treated with 7.5% admixture of (1CKD:1B:1OC) with that of the untreated tailings. It can be seen from Figure 3-21 that the shear strength of the treated soil is substantially higher than that of untreated tailings due to the cementitious bonds established by the additive used. The 7.5 % admixture of (1CD:1B:1OC) treatment resulted in substantial cohesive bonds that were manifested by cohesion intercept of 184 kPa, considering a straight line curve fit as shown in Figure 3-21. The 7.5% (0.45CKD: 0.45B: 0.1 OC) had little to no effect on the shear strength envelope other than 11kPa increase in cohesion. The cohesion gain in this case is attributed to higher dry density and not to cementitious bonds.

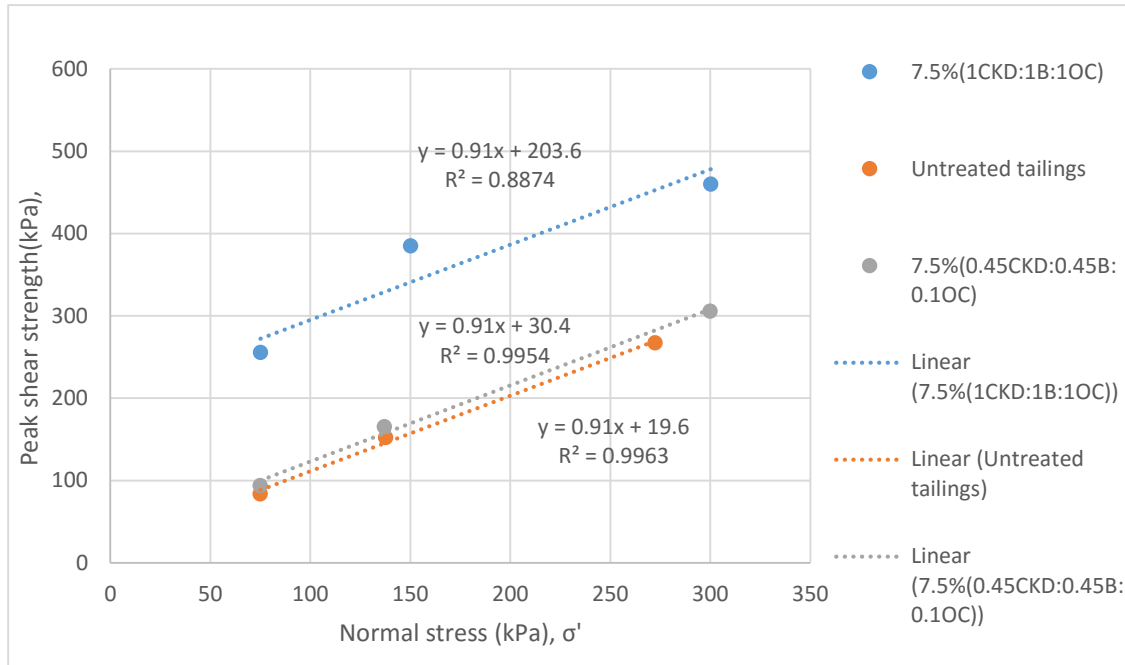


Figure 3-21 Shear strength envelope of both treated and untreated tailings.

### 3.7.2.2 Oedometer Test Results

Figures 3-22 and 3-23 compare the results of the consolidation tests for the treated and untreated tailings. It can be noted from Figure 3-22 that the effect of (0.45 CKD: 0.45 B: 0.1 OC) admixture of 7.5% by total weight did not increase the stiffness of the treated tailings. This could be attributed to two reasons: the bonds were weak and intangible; or the bonds collapsed when the effective stresses increased beyond 50kPa. Though, we can see from the shear strength envelopes (Figure 3-21) of untreated and treated tailings (0.45CKD:0.45B:0.1OC) that the cohesion of the treated material is higher than that of untreated by around 11kPa. This increase in cohesion is attributed to the “quasi”-over-consolidation that happened because of the addition of the admixture to the tailings. The acquired cohesive forces were not sustained in water environment as can be noted from Figure 3-21. However, substantial improvement was achieved when the tailings were treated with the 7.5 % admixture of (1CD:1B:1OC). The volumetric



strains were drastically reduced from the average strain of 17% to almost 2.21% as can be noted from Figure 3-23. This huge reduction is attributed to the particles bonding due the addition of cement at a higher percentage.

On the other hand, the application of the treatment increased the solids in the slurry volume, reducing the water content and void ratio, and thereby increased the dry unit weight. This situation has led to the tailings sample being of lower void ratio than the untreated tailings at any normal stress which eventually created a stronger material. This also has led to a lower hydraulic conductivity as seen from Figures 3-24 and 3-25. Finally, the tailings properties used in modelling, for untreated and 7.5% treated tailings, are shown in Table 3-6.

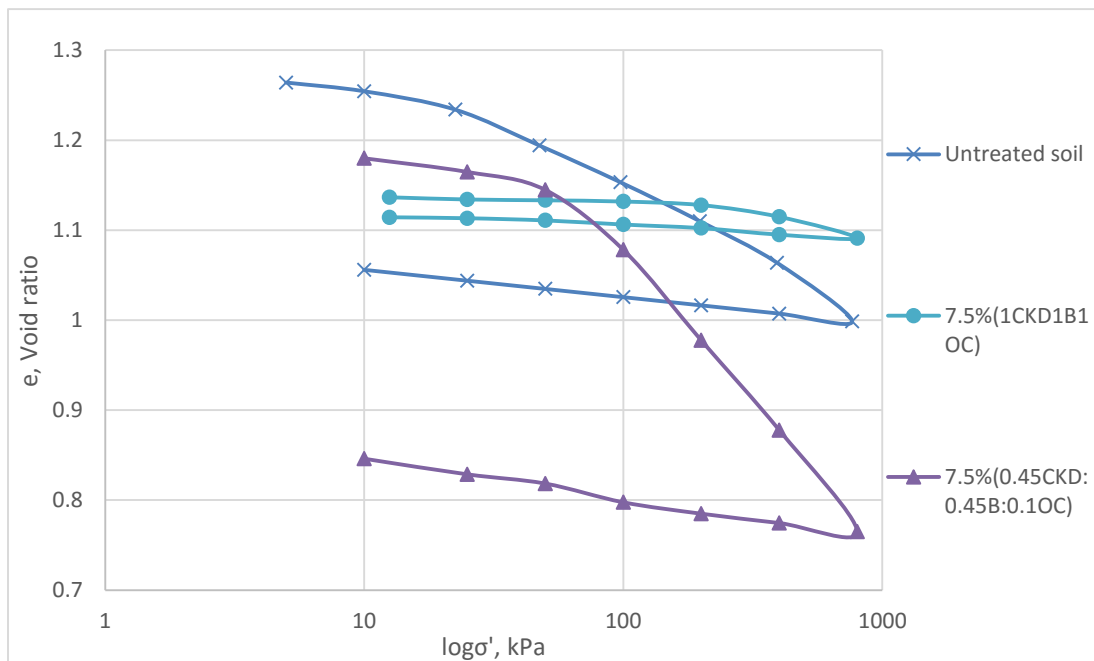


Figure 3-22  $\log \sigma'$  curves of treated and untreated tailings.

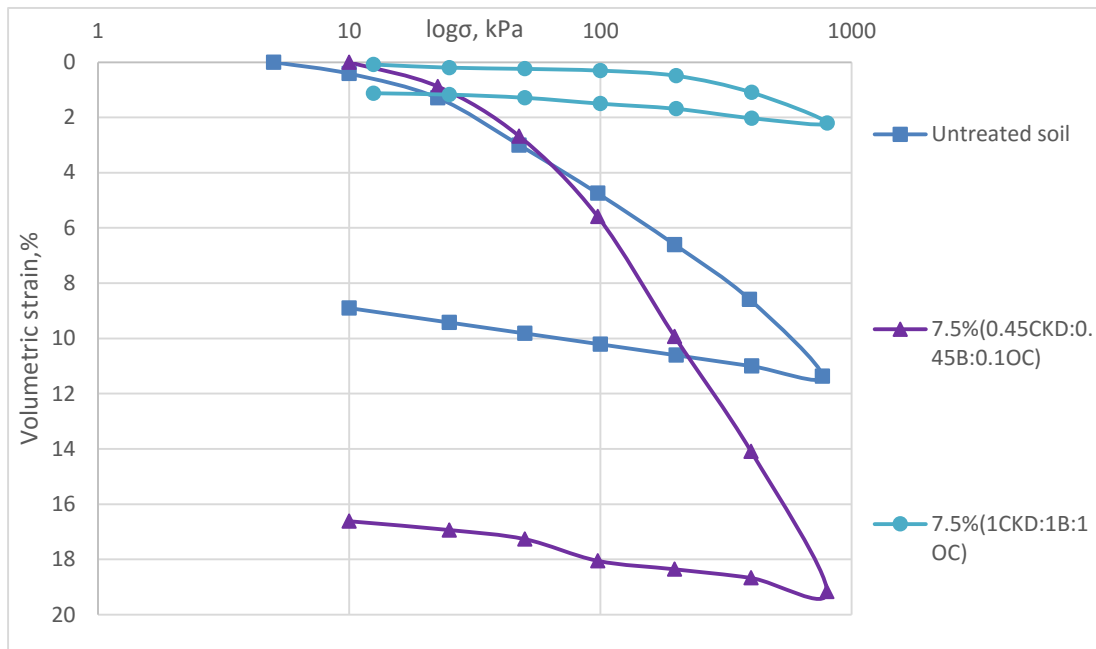


Figure 3-23 Volumetric strain-logarithmic effective stress relationship of all treated and untreated tailings.

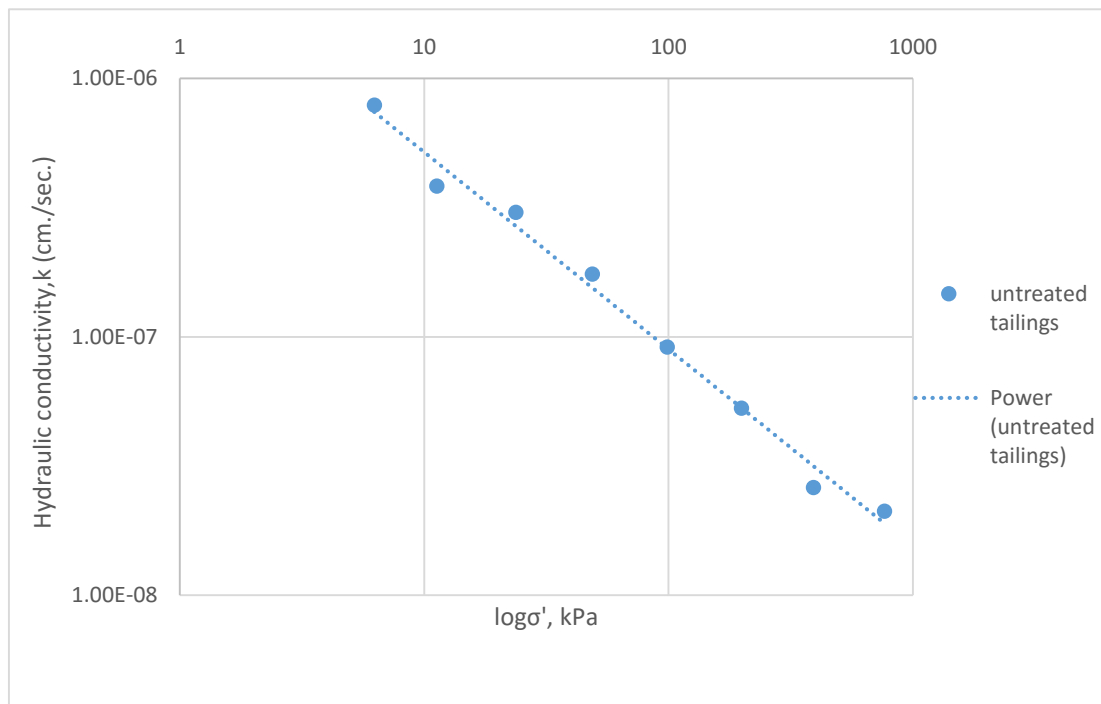


Figure 3-24 hydraulic conductivity of the untreated tailings versus effective stress.

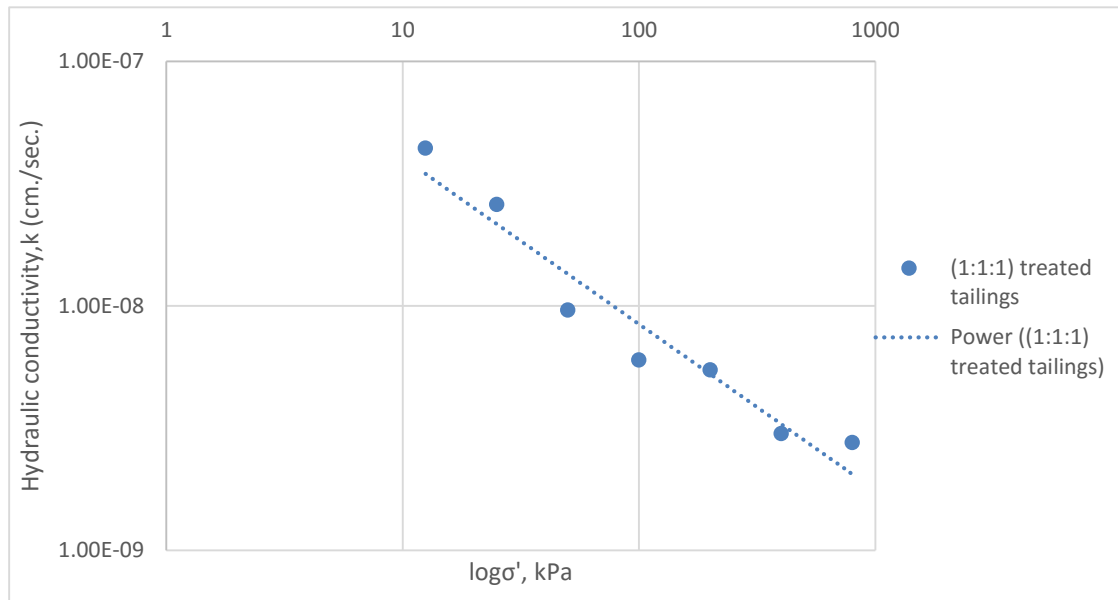


Figure 3-25 Hydraulic conductivity- effective stress relationship for (1CKD:1B:1OC) treated tailings.

From Figure 3-24 and Figure 3-25, it is proved that the addition of admixtures resulted in a reduction, around one order of magnitude, in the hydraulic conductivity of the treated tailings compared to those of untreated tailings due to the smaller initial void ratio of the treated tailings. Hydraulic conductivity chosen for modeling was selected based on the average effective stress anticipated in the field in the numerical modelling part of the current study.  $E_{oed}$ , was taken to be the secant modulus between 100-400 kPa based on the effective stresses anticipated in the field as well. Untreated tailings were assumed to be normally consolidated with effective cohesion of zero. The cohesion amount of 19.6 kPa exhibited by the untreated tailings was also subtracted from the effective cohesion of treated tailings.

*Table 3-6 Tailings properties*

Material	C'(kPa)	$\Phi'$	E <sub>oed</sub> .(kPa)	k (cm/sec.)
Untreated tailings	0	42	9400	9.12357E-08
Treated tailings (1:1:1)	184	42	30284	6.01838E-09

### 3.8 Quality Control (QC) and Quality Assurance (QA)

It is highly advised when adopting this method that both Quality Control and Quality Assurance programs are established. Quality control concerns the quality and the quantity of the binder. The efficiency of the mixing equipment in terms of the optimum rotational cycles, speed of one cycle is to be verified by the mine operator to ensure the bench scale results exist in the field condition. The QC should be carried out at the very start in laboratory and should address several aspects such as the binder type, binder percentage and W/binder ratio. On other hand, QA is carried out to ensure that the construction parameters fulfill the bench scale parameters used for design (Madhyannapu et al., 2010; Bertero and Valagussa, 2012). Field tests, e.g. geophysical tests, loading tests and penetration tests should be conducted to ensure that the treated ground meets the design specifications.

### 3.9 Summary

This chapter describes an experimental investigation to improve the tailings strength and deformation characterisations. The main motive for the research effort was a number of failure

incidents of tailings impoundment that have had impacted both humans and the environment.

The consequences of such failures are long-term and mostly severe. In Canada, the failure of Mount-Polly gold and copper tailings dam is a good example on how the failure incident could negatively influence the TSF surroundings. It has released ten millions of cubic meters of tailings that could be rich in hazardous materials to the surrounding environment.

The experimental program involved treating mine tailings with different traditional and non-traditional additives to improve their engineering properties. The improvement was assessed in terms of deformation shear strength parameters. Based on the results of the experimental investigation, the following conclusions may be drawn:

- Traditional and non-traditional additives are used, tested and optimized.
- Traditional additive of recycled gypsum with cement kiln dust (1:1) was proven to be the best tested proportion in terms of adding cohesive bonds to the non-cohesive tailings and by increasing its stiffness. However, it was found that soaking in water undid the improvements.
- Adding ordinary cement to the admixture at the proportions 1CKD: 1B: 1OC prevented the adverse impact of water. It was found that the admixture at 7.5% by total weight has substantially improved the shear strength parameters. It has added a cohesion of 184 kPa (cohesion of untreated and treated were reduced by 19.6kPa since untreated exhibited this amount of cohesion believed to have resulted from overconsolidation of DS test's specimen).

- Important laboratory observations can be helpful in the numerical models used by designers of tailings dams. For example, the change of shear strength parameters as the confining pressure increases should be considered in design.

In field applications, sufficient curing time should be allowed in order for the material to gain its strength. At least 7 days of dry curing should be allowed to avoid the adverse impact of water and to enable most of the shear strength gain before loading the treated tailings in the staged construction. Having said that, tailings that need water cover to prevent oxidation and the associated acid mine drainage (AMD) may not be favorable places to use this method. Other methods such as dewatering and adding downstream's berms will make a more stable dam as indicated from the direct shear test's results.

# Chapter 4

## **“Numerical Modelling”**

# **NUMERICAL INVESTIGATION OF EFFECTS OF TAILINGS TREATMENT ON STABILITY OF TAILINGS DAMS**

## **4.1 Introduction**

Numerical modeling offers an efficient and often reliable tool to tackle complex engineering problems. It can be used to evaluate important engineering parameters related to both the construction and design of geotechnical structures. Finite element analyses (FEA) can be used for versatile geotechnical problems such as: staged construction, seepage problems, consolidation, foundation engineering and slope stability problems. In particular, finite element analysis is used extensively for the analysis and design of dams and tailings dams.

The concept of FEA is to subdivide large domains into small elements which are composed of nodes at which some boundary conditions exist. With these boundary conditions, such as water, stress, displacement boundaries, equations are solved to estimate stress and strain fields and to integrate the results for more general outcomes such as maximum displacement, maximum stresses and maximum bending moments. Geotechnical engineers use specialized finite element analysis packages for the analysis of geotechnical structures. Some of these packages are based on the concept of FEA and some are based on the concept of finite difference. The list of commercially available programs that are widely used by geotechnical engineering practitioners include: PLAXIS (both 2-dimensional and 3-dimensional) (Ormann et al., 2013; Zardari, 2013), FLAC, both 2-dimensional and 3-dimensional (Firoozi et al., 2014), Geo-Studio (Government of British Columbia, 2015). CLARA-W (Nikbakhtan et al., 2007), and Slide (Ozcan et al., 2012). Some of these programs allow different constitutive models to simulate the behavior of different soil types. They also allow modeling the response considering both drained and undrained



conditions. Most programs enable mesh refinement to increase the accuracy of results at certain location(s) where higher stress concentration is anticipated. Furthermore, simulation tools are incorporated to enable displaying the expected final deformed shape, which helps identify failure mechanisms and sometime aid in indicating errors of assigning some boundary conditions or stresses. Final stress fields can also be established to spot-check the results at predetermined locations where the outcome is known in advance. This can help the designer to re-adjust the construction sequence or material used.

### 4.2 Literature Review

Several case studies are available in the literature where numerical modeling was used for the analysis and design of tailings dams. For example, Ozcan et al. (2012) studied the downstream slope stability of a copper Turkish tailings dam to assess the proposed capacity increase. They utilized the program Slide 2D, which uses limit equilibrium method for stability analysis. Ormann et al. (2013) investigated the stability of a Swedish tailings dam considering the proposed construction/raising sequence. They studied the effect of the staged construction on the safety factor using PLAXIS 2D. They concluded that the safety is affected by the cycles of loading/excess pore water pressure generation and the associated consolidation and proposed a new method of stabilization using rock-fills. Yin et al. (2011) modelled the slope stability of a laboratory physical model to study the influence of staged construction of the dam using Slide 2D. Seneviratne et al. (1996) extended the work of Gibson (1967) and developed a consolidation program, “MinTaCo”, to estimate the self-consolidation settlement of tailings dam. They evaluated the self-weight consolidation that can happen at different filling and evaporation rates. Employing the program MinTaCo, they investigated optimization of the available storage by taking into consideration the evaporation and filling rates.

Beier et al. (2015) employed a TMsim model, which is coupled with Excel macros and FSConsol to investigate the use of certain tailings management plan. Several results can be obtain at the end of modelling including: available storage volume, required impoundment storage volume, available recycled water volume and quality and strength gain trajectories within the deposit (Seneviratne et al., 1996; Beier et al., 2015). Barnekow et al. (1999) used FSConsol program, which is a consolidation program based on the Finite Strain Theory by Gibson et al. (1967), to study the consolidation settlement of WISMUT tailings dam in Germany, which was constructed using the centerline method. They investigated the excessive self-weight settlement, amounting to 0.2m per year for an average thickness of 45m. They underscored the importance of characterizing the compressibility of the tailings slimes and the knowledge of its filling history in order to define the final shape of the dam and to estimate the capping layer thickness. They evaluated variation of  $e$  with depth and verified the results by means of field testing, which confirmed the program accuracy.

Numerical modeling can also help gain insights on seemingly intuitive concepts that may not actually be true. For example, the common wisdom is that the coarser material always entails higher drainage and thus should not form the outer layer of a soil cover. However, as can be seen from  $k$ -matric suction relationship shown in Figure 4-1, the fine soil can be more conductive at high matric suction. This means the infiltration rates, and thus the matric suction, are significant considerations for unsaturated soil mechanics. Some researchers have pointed out to this problem of soil cover and explained it in the context of volumetric water content versus suction or hydraulic conductivity versus suction relationship (Newman et al., 1997). This situation can be examined using FEA by assigning the unsaturated hydraulic conductivities to the different soil layers to predict the drainage pattern with altering coarse/fine covers positions.

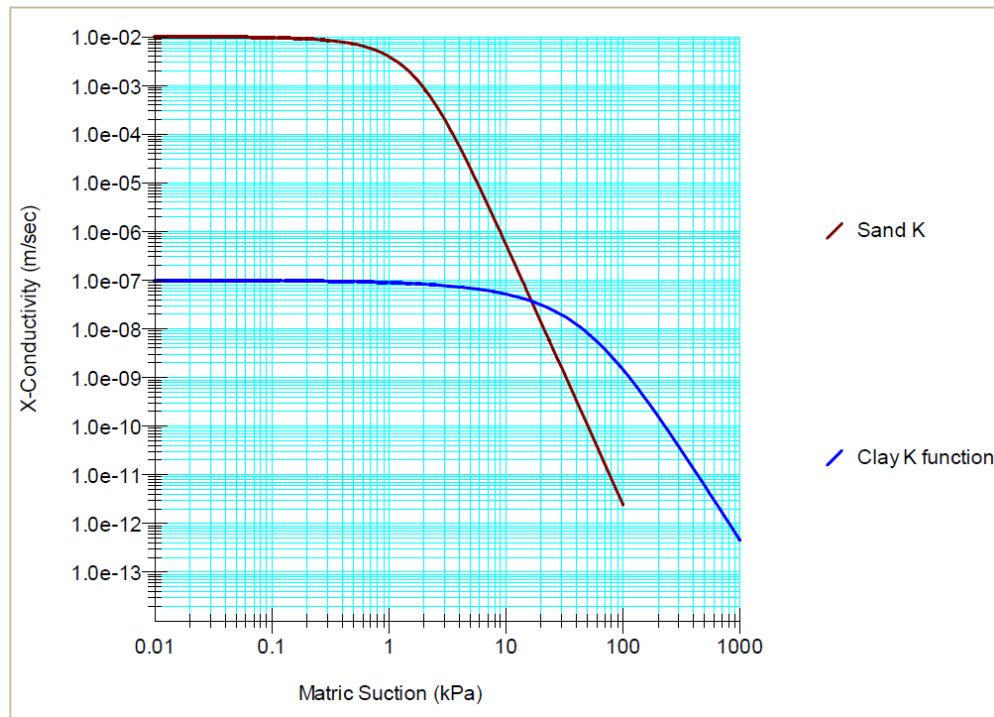


Figure 4-1 hydraulic conductivity-matric suction (kPa) relationships of both fine and coarse soils (Seep/W tutorial, 2012)

### 4.2.1 Slope Stability Analysis

There are two main approaches to conduct slope stability analysis. The first and earliest method is limit equilibrium (LE) approach, which was presented by Petterson for the analysis of the possible failure mechanisms of Stigberg Quay in Gothenberg, Sweden (Krahn, 2002). The main assumptions in the earliest method developed, Swedish circle, is that the slip surface is circular and the soil or rock mass's shear strength is given by  $\tau = s_u$ , i.e., neglecting the frictional resistance. This approach was further enhanced and extended since then by many researchers (e.g. Janbu, 1954; Bishop, 1955; Morgenstern-Price, 1965; Morgenstern, 1967; and Fredlund-

Krahn, 1970). The LE analysis can generally yield satisfactory results for the overall safety factor (SF) but is considered to be lacking in the following aspects:

- LE is a plastic analysis and thus treats the slip mass as a rigid body
- Strain field is not established
- SF is constant for all slices (no information about localized plastic strains).

The finite element analysis provides an alternative approach for the LE method, which can help alleviate its shortcomings. In FE based stability analysis, stress and strain fields are established, i.e., information about local strain, especially plastic strains becomes available. This information can help identify the failure mechanism and track progressive failure. In addition, overstressed regions can be assigned lower shear strength parameters, i.e., residual strength parameters, to account for the strain softening behavior exhibited by some clayey soils.

There are two approaches to calculating the safety factor in the FE based stability analysis: in-situ stresses based SF and strength reduction technique. Strength reduction technique, such as Phi-C technique in PLAXIS 2D, resembles the limit equilibrium method in that the strain after “failure” is not realistic. However, the FE computed strains without strength reduction can be accurate and useful.

### 4.2.2 Application of Soil Improvement to Tailings Dams

Ground improvement techniques are used to enhance the stability of slopes and dams when analysis demonstrates potential instability. Nikbakhtan et al. (2007) presented a case study of the efficacy of jet grouting in increasing the slope stability. Their parametric study was done using the finite element program CLARA-W. They compared the factor of safety in the staged

construction of the Shahriar Dam, Iran using 2D and 3D analyses. Their results proved the effectiveness of soil treatment. Shu-wei et al. (2013) and Poulos (1995) examined the usefulness of micro-piles and piles in increasing slopes' stability. Their results indicated that piles can be effective stabilizing measure to counteract the sliding forces. For the mine tailings dams built using the wet method, consolidated tailings, or composite tailings, (CT) and rock-fills have been used to increase the whole dam's stability. However, no attempt, to the author knowledge, has been done to "mix" the wet tailings with additives before disposal has been reported so far.

In the current research, traditional additive admixture of recycled Gypsum and Cement Kiln Dust (CKD) compelling performance with different soil types stimulated their selection to stabilize mine tailings impoundments built using the upstream method of construction.

### **4.3 Objectives and Scope of Work**

The objective of this work is to demonstrate the effectiveness of soil treatment for stabilizing mine tailings dams. A case study involving numerical modeling of a tailings dam using the finite element analysis was employed for verification of the numerical model developed in the current study. The verified numerical model was then utilized to analyze the stability of a tailings dam constructed using the upstream method considering the properties of the tailings material examined in Chapter 3. The performance of the tailings dam constructed using treated tailings with 7.5% admixture of (1 CKD: 1B: 1OC) as discussed in Chapter 3 was analyzed. The effects of tailings treatment on the stability of the dam and its settlement were evaluated.

### **4.4 Methodology**

The numerical investigation phase was conducted using the FE program PLAXIS 2D, which allows analysis of staged construction and therefore enables accounting for excess pore pressure

(EPP) development and subsequent dissipation and consolidation behavior of the staged construction used in upstream tailings dam. To evaluate the effectiveness of the proposed solution to enhance mine tailings stability, the results of the soil strengthening obtained from the experimental program reported in Chapter 3 were used in simulating the treated tailings. The numerical model was constructed following the procedure of staged construction and the stability of the tailings dam was evaluated at each stage of construction. The numerical model was verified through the analysis of a well-documented case study of Aitik upstream mine tailings dam in Sweden whose geometry and soil properties are reported by Ormann et al. (2013), Pousette et al. (2007) and Bjekleiv (2005). This case study was selected in particular due to its complexity and authenticity.

PLAXIS 2D is a renowned geotechnical programs that allows auto-meshing and easy defined boundary conditions. For problems involving slope stability, it allows four computation types: plastic, consolidation, safety and dynamic analyses. The program has a library of constitutive relationships that can describe different soil behaviour, including: Mohr-Coulomb (MC), Modified Cam Clay (MCC), Plaxis hardening soil (HS), Soft Soil (SS), and Soft Soil Creep (SSC) models. Duncan (1996) conducted an extensive review on the different constitutive models employed for the analysis of earth dams. He concluded that every model has its own advantages and limitations. However, he stated that, though they have the limitation that they are more complex, the Elastoplastic and Elasto-viscoplastic stress-strain relationships have the advantage that they model more realistically the behavior of soils close to failure, at failure, and after failure (Duncan, 1996). Similar investigations were conducted more recently to examine different available constitutive models (e.g. Labuz and Zang, 2012; Ti et al., 2009; Zheng et al., 2005; Griffiths and Lane, 1999). Most studies demonstrated that the Mohr-Coulomb (MC) is

suitable for simulating the behavior of soil in slope and dam stability problems. Therefore, it was used in the current study to simulate the behavior of both treated and untreated tailings.

The constitutive model used herein is a simple elastic- perfectly plastic where the soil shear strength is represented by the MC failure criterion. In addition to material's unit weight, this model requires only five parameters:  $\phi'$ ,  $c'$ ,  $\psi'$ ,  $E$  and  $\nu$ . The tailings are considered to be silty material, and are assumed to be non-dilative. The strength and deformation parameters of the treated and treated tailings were established from the Direct Shear (DS) and Oedometer tests. The Poisson's ratio and dilation angle,  $\psi'$  are assumed to be 0.33 and  $0^\circ$  for all soil layers used in the numerical model. In addition, the hydraulic conductivity for the tailings was established from the results of the Oedometer tests. The hydraulic conductivity was estimated based of the following equation (Das, 2006):

$$K = c_v m_v \gamma_w \dots\dots\dots (4-1)$$

Since the Oedometer is only vertically draining, it was assumed that hydraulic conductivity in both vertical and horizontal directions,  $k_y$  and  $k_x$ , are equal.

### 4.4.1 Numerical Model

The numerical model was established for the purpose of assessing the stability of Aitik upstream mine tailings dam whose geometry and soil properties are well-documented (Ormann et al, 2013; Pousette et al., 2007; Bjekleiv, 2005). The model analyzed the construction of the dam which involved 11 construction stages (raises) executed over the course of 11 years. Two dimensional plane strain model of the tailings dam was formulated using the required information reported in the literature on its geometry and soil properties. The model is then used

to explore the effect of soil improvement on the dam behavior through the different construction stages.

### *4.4.1.1 Case Study of Aitik Tailings Dam*

Aitik is an open pit copper mine located close to Gällivare in northern Sweden. The mining activities at Aitik started in 1968 with the 2011's production of this mine reaching 31.5 million tons of ore. The tailings dam is 450 m wide and 70m in height. The tailings are pumped to the tailings disposal area and discharged by spigotting from the dam crest. Figure 4-2 shows the impoundment which covers a 13 km<sup>2</sup> area and is limited by the topography and four dams: A-B, C-D, E-F (including E-F2 extension), and G-H. The settling pond is situated downstream of dam E-F. View of dam E-F is depicted in Figure 4-3.

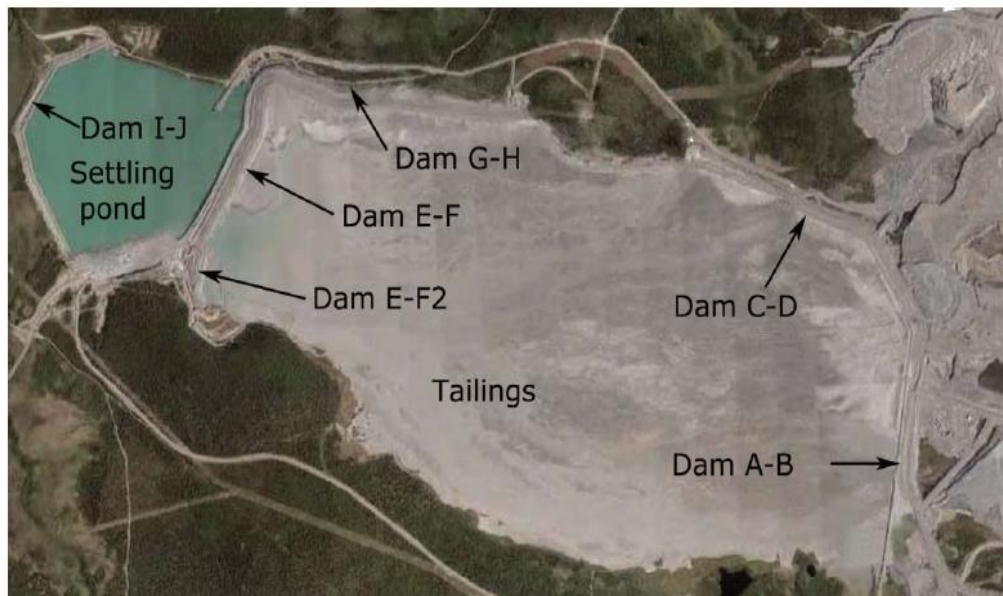


Figure 4-2 Areal view of Aitik tailings impoundment (After Ormann et al., 2013)





Figure 4-3 View on the modelled section, E-F, of the upstream built dam

(After Ormann et al., 2013)

The section E-F of the tailings dam failed in 2000 over a length of 120m. Consequently, 2.5 million cubic meters of water leaked to the settling pond. Therefore, the leaked water had filled up the settling pond whereby the water flowed to nearby rivers. No definite conclusion concerning the cause of the failure was reached (Ormann et al. 2013). The finite element analysis in this case study has been performed on a section of dam E-F, shown in Fig. 4-3.

The failure of Dam E-F had serious consequences for humans and the surrounding environment. In addition, its failure may lead to a failure of dam I-J, which is located downstream of dam E-F and the settling pond (Fig. 4-2). Dam E-F was constructed using the upstream method. The different material zones are presented in Figure 4-4. Material zones 2, 3, 5, 6, 7, and 8 (Fig. 4-4) represent the coarse tailings particles. These material zones can be classified as silty sands according to the unified soil classification system.

Field testing campaign had been launched to retrieve samples for lab testing to determine the geotechnical parameters that were required for the analysis of the dam stability. The numerical model was established using Plaxis 2D and employing 15 noded triangular elements, which give a fourth order interpolation on displacement (Brinkgreve et al., 2015). The elastic-perfectly plastic model and MC failure criterion were used to assess the variation of dam stability should the proposed plan of construction be followed. Standard fixities in PLAXIS 2D were utilized, including: fixed base to constrain horizontal and vertical movements and constrained horizontal displacements at the left vertical boundary. Water was allowed to seep in all direction except the left and bottom boundaries (Ormann et al., 2013). Each layer was constructed in 10 days and was allowed to consolidate for 355 days. This short period of construction generates excess pore pressures that dissipate with time. Not all the EPP is expected to have dissipated by the end of construction due to the existing hydraulic boundaries and low hydraulic conductivities of the materials. Therefore, the stability of the dam is expected to decrease during construction period and increase afterwards since the effective stresses increase at the same rate at which the EPP decays.

Duncan (1996) stated “For embankments and multistage loading conditions where the loading results in increased stresses in the soil, the short term condition is critical. This is because these types of loads result in positive changes in pore pressures, and, as these positive excess pore pressures dissipate over time, the effective stresses and the strength of the soil increase”.

Therefore, consolidation analyses along with safety analyses were conducted in order to establish safety factors fluctuation of the Aitik tailings dam with loading/consolidation cycles of 11 years long. To enhance the stability of the dam and to increase its safety factor, the use of rock-fills to stabilize the tailings dam was investigated by Ormann et al. (2013). Ormann et al. (2013) have

concluded that the rock-fills option utilized was successful in enhancing the stability of the upstream built tailings dam.

In the current study, it is proposed to use improved tailings in lieu of rock-fills to enhance the overall dam stability. The properties of the treated and untreated tailings determined from the experimental study reported in Chapter 3 are used in the numerical model. The program PLAXIS 2D was utilized for the numerical modeling and the Phi-C reduction technique was employed to estimate the safety factor. Both the internal angle of friction and the cohesion were reduced at the same proportion to weaken the soil and bring it into a non-equilibrium state. This method is sometimes termed strength reduction technique (Griffiths and Lane, 1999). The safety factor in this comes is defined as:

$$SF = \frac{C'_{Actual}}{C'_{Reduced}} = \frac{\Phi'_{Actual}}{\Phi'_{Reduced}} \dots\dots\dots (4-2)$$

Figure 4-4 shows the details and dimension of the model developed to simulate the Aitiki dam. The geometry consists of 8 rock-fill layers used for optimization work by Ormanne et al. (2013). Nine optimization analyses were conducted. In each analysis, either one or all the rock-fills utilized was/were activated in order to stabilize the potential slip surface. In other words, in each of the 9 cases of optimization a number of these layers was activated, varying from 1 to 8, in order to increase the slope stability of the dam. Figure 4-6 shows the symbols designated to each rock-fill layer (P, Q, R, S, T, U, V, &W). Table 4-1 shows the details of each case (from case I to IX). For instance, Case V has four rock-fills layer out of eight (P, Q, R, and S) activated during 2<sup>nd</sup>, 4<sup>th</sup>, 5<sup>th</sup> and 6<sup>th</sup> years of construction, respectively.

Figure 4-5 shows the case I in Ormann et al. (2013) model, only the existing rock-fills activated.

Figure 4-6 and Table 4-1 show the geometry of the rock-filling utilized in their study and the details of the activated layers for each case of the optimization analyses.

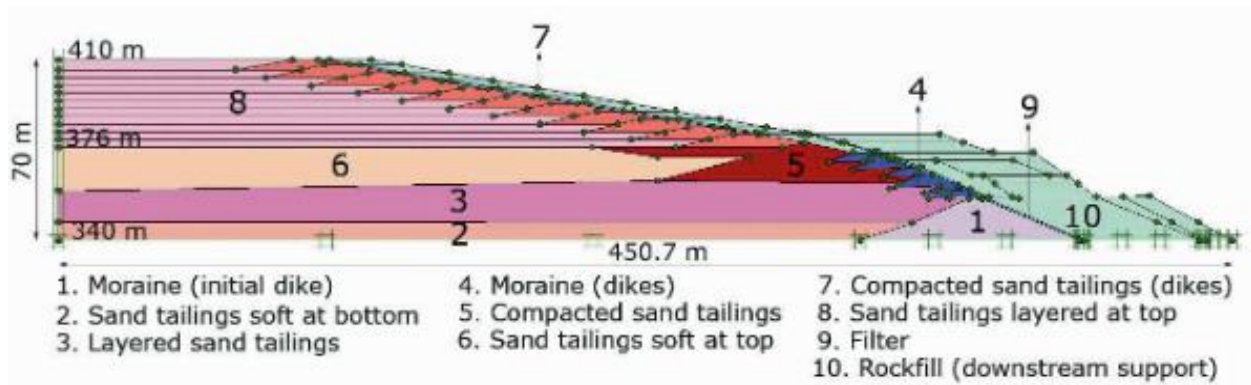


Figure 4-4 Cross section of the dam to be replicated with all the proposed rock-fill banks.

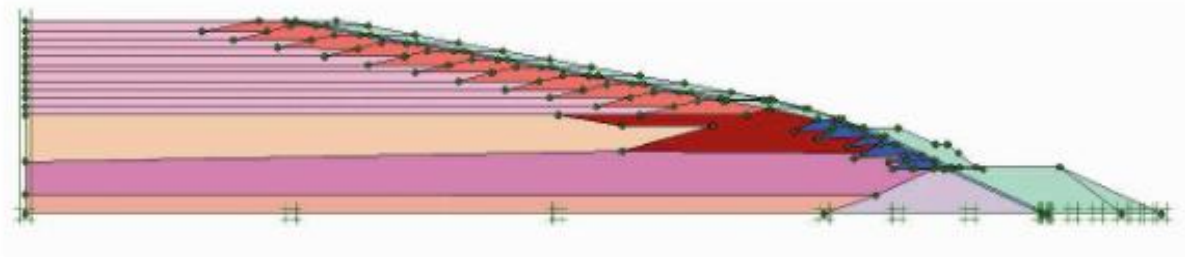


Figure 4-5 Case I is shown which the case if no rock-fill layer is added except the existing ones (light green color).

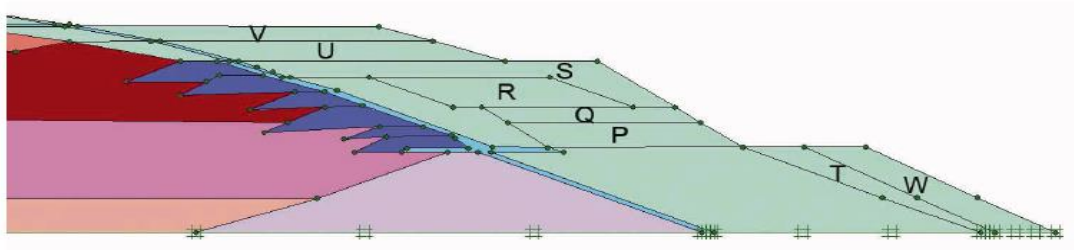


Figure 4-6 Details of the rock-filling option utilized (light green color).

Ormann et al. (2013) conducted the analysis of the model where the 11 lifts shown to the top left corner of Figure 4-4 were successively added over the course of 11 years. Each layer was constructed in a period of 10 days followed by a period of 355 days for consolidation. Then the following layer is constructed over 10 days followed by 355 days of consolidation. The same procedure was then repeated for all the following cases. Afterwards, the safety factors were drawn (2 FOS for each layer; one once the layer is constructed and the other after the 355days end) with the number of years (Figure 4-7). Once that was accomplished, the analyses were re-run but with adding varying number of rock-fill banks downstream as shown in Figure 4-6 and Table4-1.

***Table 4-1 the number of activated rock-filling layers corresponding to different cases of optimization carried out, After Ormann (2013).***

Case No.	Raising 1	Raising 2	Raising 3	Raising 4	Raising 5	Raising 6	Raising 7	Raising 8	Raising 9	Raising 10	Raising 11
I	-	-	-	-	-	-	-	-	-	-	-
II	-	P	-	-	-	-	-	-	-	-	-
III	-	P	-	Q	-	-	-	-	-	-	-
IV	-	P	-	Q	R	-	-	-	-	-	-
V	-	P	-	Q	R	S	-	-	-	-	-
VI	-	P	-	Q	R	S	T	-	-	-	-
VII	-	P	-	Q	R	S	T	U	-	-	-
VIII	-	P	-	Q	R	S	T	U	V	-	-
IX	-	P	-	Q	R	S	T	U	V	W	-

Since the objective of this thesis is to show how the safety factor is increased for a certain improved tailings over the un-improved tailings, three cases from those shown in table 4-1 were selected for verification purpose (Case I, V and IX).

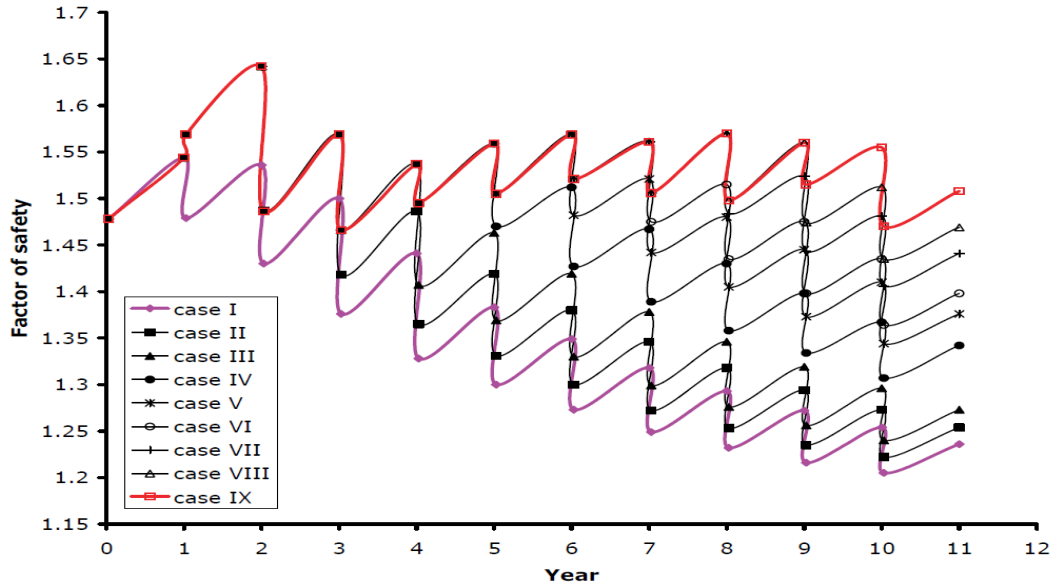


Figure 4-7 Safety factors versus number of years (Ormann, 2013).

#### 4.4.1.2 Verification of Numerical Model

Prior to using the numerical model for evaluating the effectiveness of the tailings treatment technique investigated in Chapter 3 for enhancing the dam stability, the numerical model was first verified against the published results of the Aitiki dam (Ormann et al., 2013). For this purpose, a simplified version of Ormann et al. (2013) model geometry was considered in the analysis. The loading, consolidation, and safety analyses were carried out similar to the original model of Ormann et al. (2013).

Figure 4-8 shows the developed simplified model. The complex geometry features and layers whose presence would not affect the accuracy were excluded. The dam body and its foundation were modeled employing 15-nodded triangular elements. Mesh refinement at highly stressed/strained zones was necessary to ensure the results accuracy. Accordingly, a series of models were developed where the mesh was incrementally refined and the results were then

compared. When the difference between the results of two consecutive models (i.e. refinements) became less than 2.5%, the most refined of the models was considered.

The standard fixities in Plaxis 2D, fixed base in all directions and the vertical boundary in horizontal direction, were used as well as closed water boundary at the left vertical and bottom ends. The material properties used for the model verification are shown in Table 4-2. In the study of Ormann et al. (2013), the excess pore pressures right after the 2<sup>nd</sup> and 11<sup>th</sup> layers were constructed and after they were allowed to consolidate were shown.

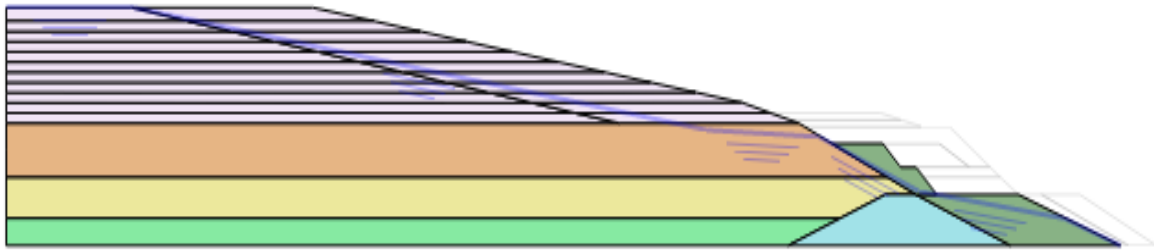


Figure 4-8 the simplified version of Ormann et al. (2013) model.

**Table 4-2 Materials properties of the replicated model (Ormann et al., 2013).**

Material type	$\gamma_{unsat}$ (kN/m <sup>3</sup> )	$\gamma_{sat}$ (kN/m <sup>3</sup> )	$k_x$ (m/s)	$k_y$ (m/s)	$E$ (kN/m <sup>2</sup> )	$c'$ (kN/m <sup>2</sup> )	$\phi'$ (°)
Moraine (initial dike)	20	22	$9.95 \times 10^{-8}$	$4.98 \times 10^{-8}$	20000	1	35
Sand tailings soft at bottom	18	18	$9.95 \times 10^{-8}$	$1 \times 10^{-8}$	9800	6	18
Layered sand tailings	17	18.5	$5.5 \times 10^{-7}$	$5.56 \times 10^{-8}$	9312	9.5	22
Moraine (dikes)	20	22	$4.98 \times 10^{-8}$	$1 \times 10^{-8}$	20000	1	37
Compacted sand tailings	16	19	$1 \times 10^{-6}$	$9.95 \times 10^{-8}$	8790	13	26
Sand tailings soft at top	18	18	$9.95 \times 10^{-8}$	$1 \times 10^{-8}$	3048	6	18
Compacted sand tailings (dikes)	16	19	$1 \times 10^{-6}$	$9.95 \times 10^{-8}$	7200	13	26
Sand tailings layered at top	17	18.5	$5.5 \times 10^{-7}$	$5.56 \times 10^{-8}$	3895	9.5	22
Filter	18	20	$1 \times 10^{-3}$	$1 \times 10^{-3}$	20000	1	32
Rockfill (downstream support)	18	20	$1 \times 10^{-1}$	$1 \times 10^{-1}$	40000	1	42



Figures 4-9, 4-11, 4-13, and 4-15 depict the EPPs for those raises from the simplified model whereas Figures 4-10, 4-12, 4-14 and 4-16 show the results from Orman et al. (2013) model.

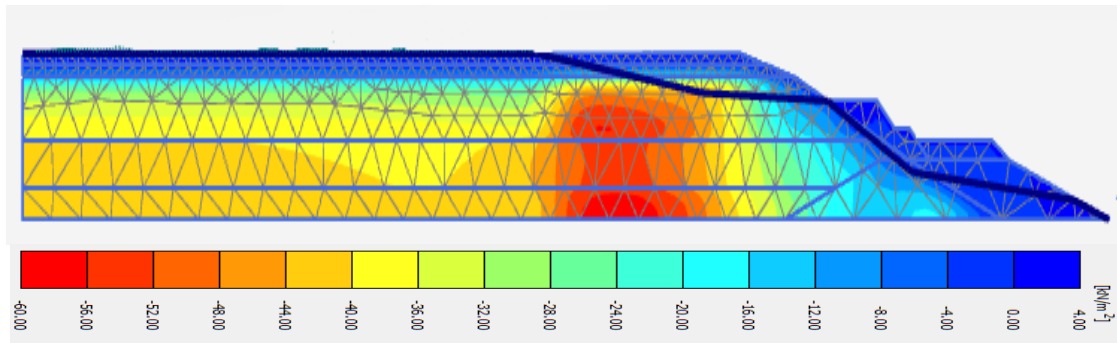


Figure 4-9 Excess pore pressure after constructing the second layer.

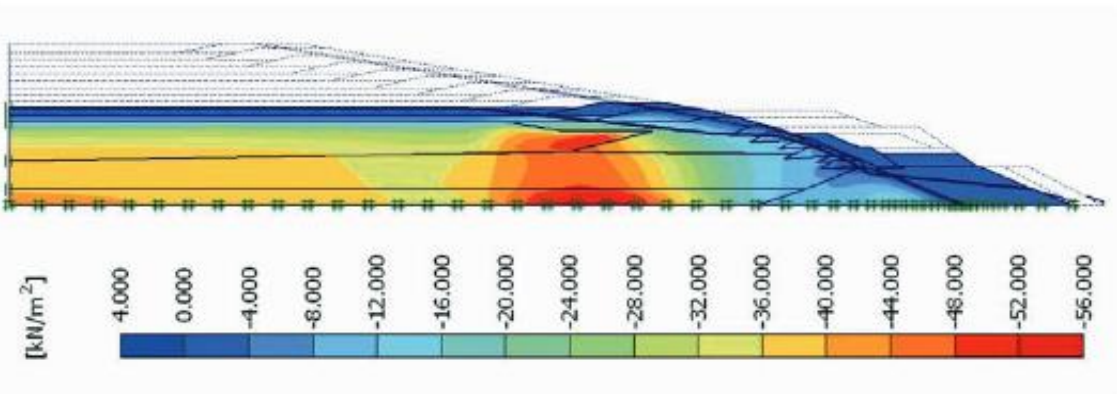


Figure 4-10 Excess pore pressure after second layer placement of the published model (After Ormann et al., 2013).

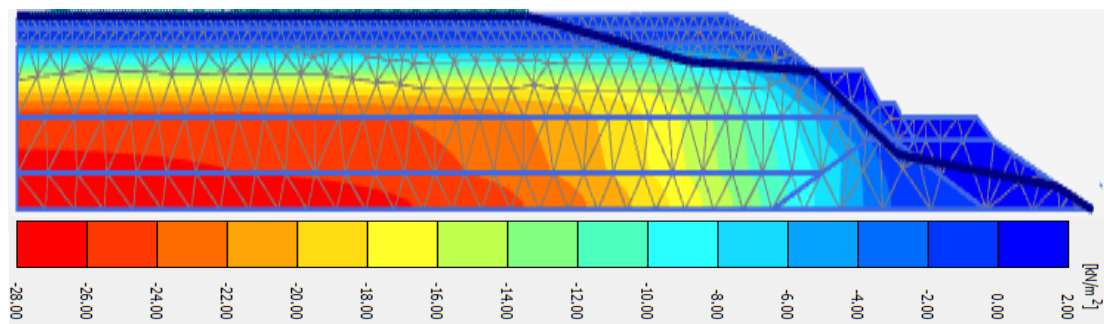


Figure 4-11 Excess pore pressure after the consolidation time for the second layer.



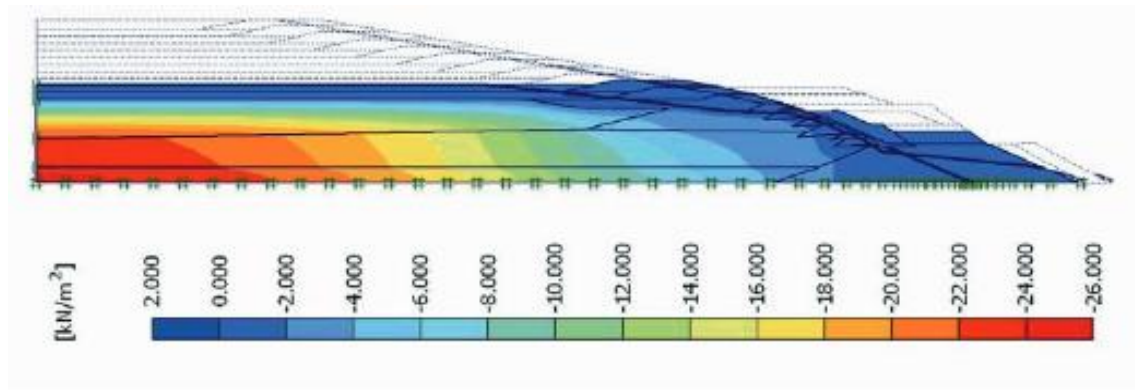


Figure 4-12 Excess pore pressure after consolidation has taken place for the second layer (After Ormann et al., 2013).

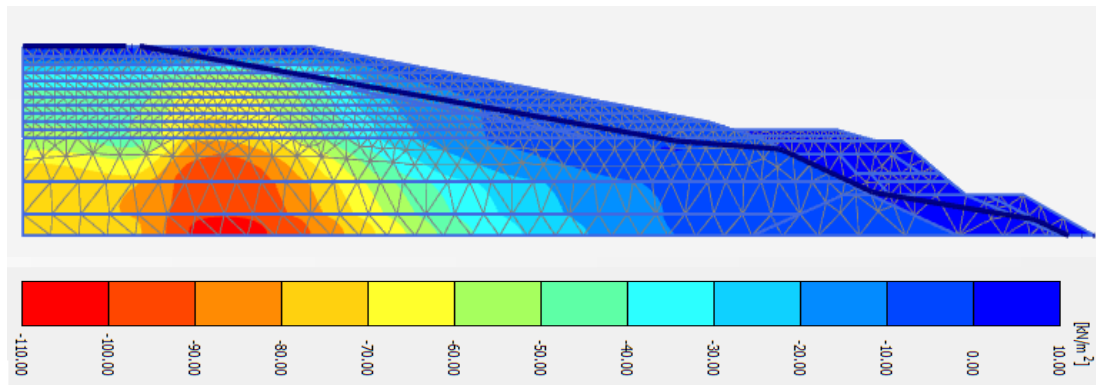


Figure 4-13 Excess pore pressure after placement of 11<sup>th</sup> rise of current study.

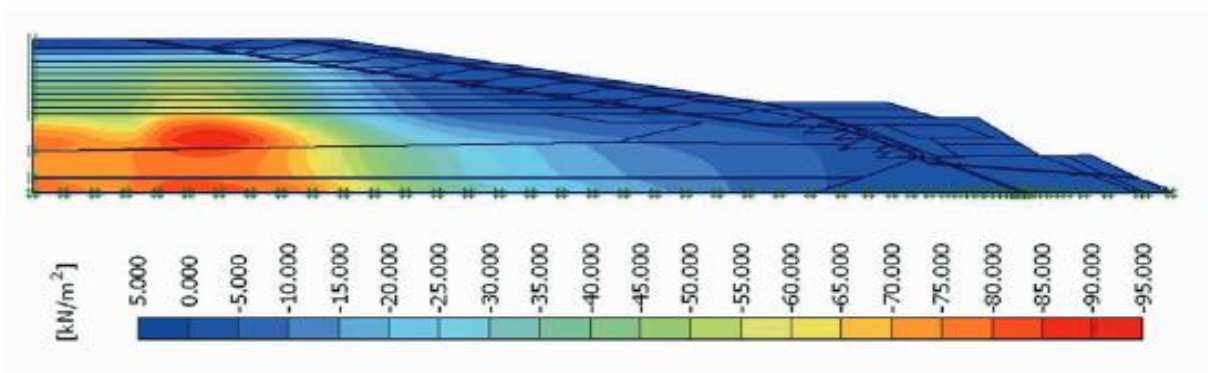


Figure 4-14 Excess pore pressure after placing the 11 lift (After Ormann et al., 2013).

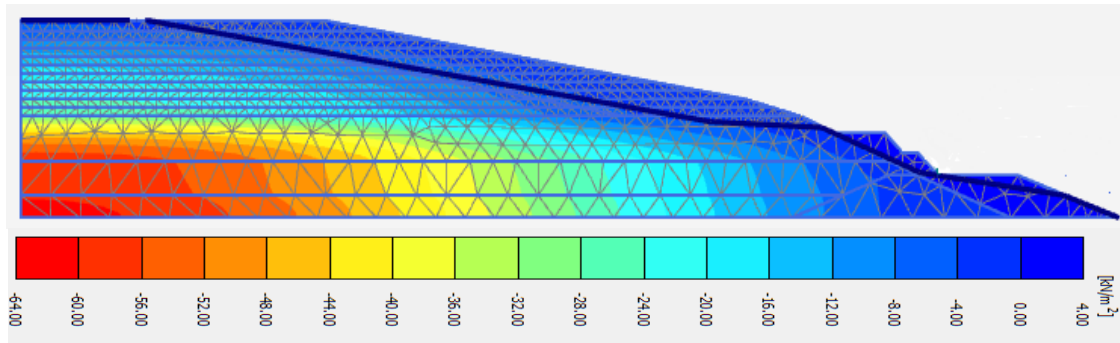


Figure 4-15 Excess pore pressure after consolidation of 11<sup>th</sup> rise.

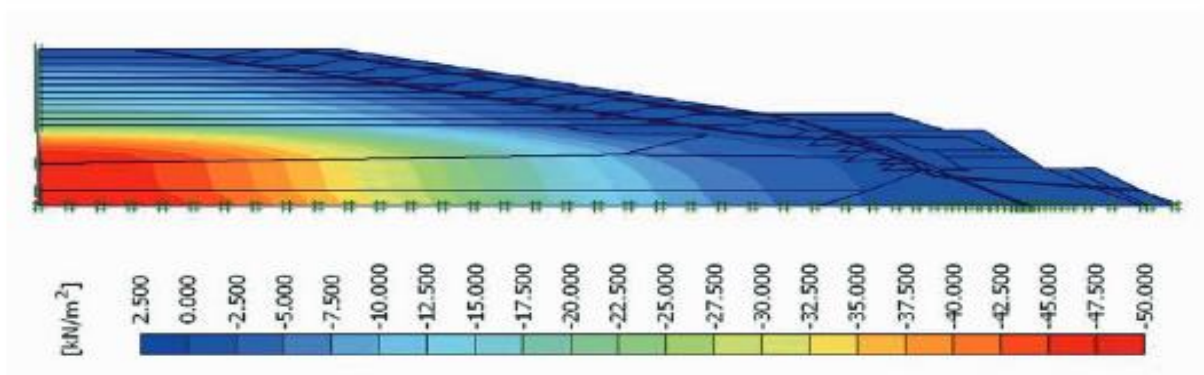


Figure 4-16 After the dissipation of the 11<sup>th</sup> lift's excess pore pressures (After Ormann et al., 2013).

Figures 4-9 to 4-16 demonstrate that the results obtained from the simplified model are in good agreement with the results obtained by Ormann et al. (2013). The excess pore pressures were all almost matching the results of Ormann et al. (2013). It should be noted that the concentration of excess pore pressure in the bottom left corner is attributed to the presence of impervious base and closed water boundary on the vertical left boundary. The EPP of the simplified model after the second layer placement was 59kPa, which compares favourably with the EPP of 56kPa predicted by Ormann et al. (2013). Same observation can be made for the EPP after consolidation occurs for the 2<sup>nd</sup> layer, in which case the difference between the two predictions was only 1kPa. For the

11<sup>th</sup> raise, EPP values were 110-59kPa and 95-50kPa right after the construction of the layer and after the consolidation, for the simplified model and for Ormann et al. (2013) model, respectively.

Figure 4-17 displays the variation of safety factor with time for both the simplified model and the model by Ormann et al. (2013). The solid squares-dotted lines in Figure 4-17 denote the results obtained by Ormann et al. (2013) whereas the hollow triangles- dotted lines are those obtained by the model developed herein. As can be seen, good agreement was established between the two models for the selected three cases (Case I, V and IX; Table 4-1 and Figure 4-6) knowing that there was difference in geometry interpretation and piezometric line location. The lower results for the first five years were because a large part of the slip surfaces developed within the 6<sup>th</sup> layer that happened to be of lower shear strength values than that of the layer it replaced (see Table 4-2 and Figures 4-4 and 4-8). The model was also verified by hand calculations where hydrostatic pore pressures, total and effective stresses estimated at the bottom left corner were compared to the results obtained by the F.E.A.

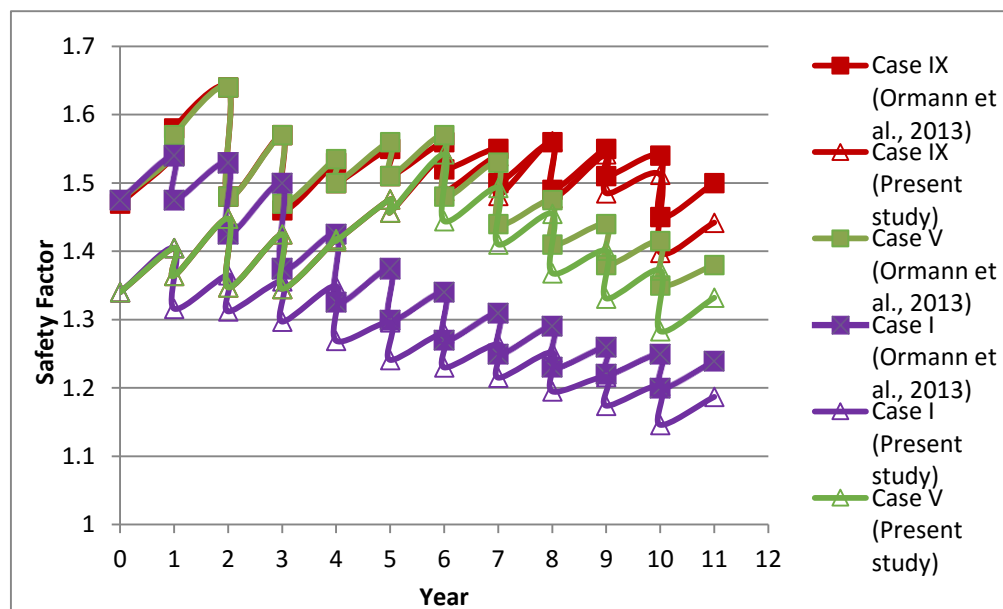


Figure 4-17 Safety factors computation versus the number of years.

### 4.4.2 Numerical Model Results for Tested Tailings

The geometry of the tailings dam model considered in this analysis is same as that shown in Figure 4-8. In addition, all layers' properties were the same as those considered by Ormann et al. (2013) except for layered sand tailings whose new assigned properties are those of treated and untreated materials determined in Chapter 3 (Table 3-7). The hydraulic conductivity used in this analysis was taken as the average hydraulic conductivity in the range of confining pressure applied in the oedometer test, while  $E_{\text{oed.}}$  was taken to be the secant modulus for applied pressure ranging between 100-400 kPa. Figure 4-18 shows the variation of SF with for all three cases: I, V and IX (Table 4-1).

#### 4.4.2.1 *Effect of Tailings Treatment on Safety Factor*

Figure 4-18 demonstrates clearly that the stability of the dam for Case I represented the worst case scenario among the cases considered. The calculated average SF for Case I, untreated, is 1.25, which is less than the required long term safety factor of 1.5 (Das, 2006; Duncan, 1996). Therefore, a soil treatment scheme is considered to enhance the stability of the dam. The analysis was repeated for the dam with identical geometry, but the tailings layers were assigned the properties of the treated tailings. The results obtained from this analysis are also presented in Figure 4-18. The calculated SF for this case increased substantially to an average value of 1.55. This significant increase in SF demonstrates the effectiveness of the soil treatment method proposed, and its suitability to enhance the stability of the upstream tailings dams. The achieved average SF of 1.55 meets the conventional requirement for long term slope stability. These results imply that the treatment scheme considered in the analysis resulted in 25% increase in the SF of the dam. Besides, Figure 4-18 shows the S.F variations for untreated tailings when rockfills

were used for two cases of Ormann et al. (2013) model. From the figure, it is shown that using the treatment scheme proposed will yield S.F much higher than when rockfills were used.

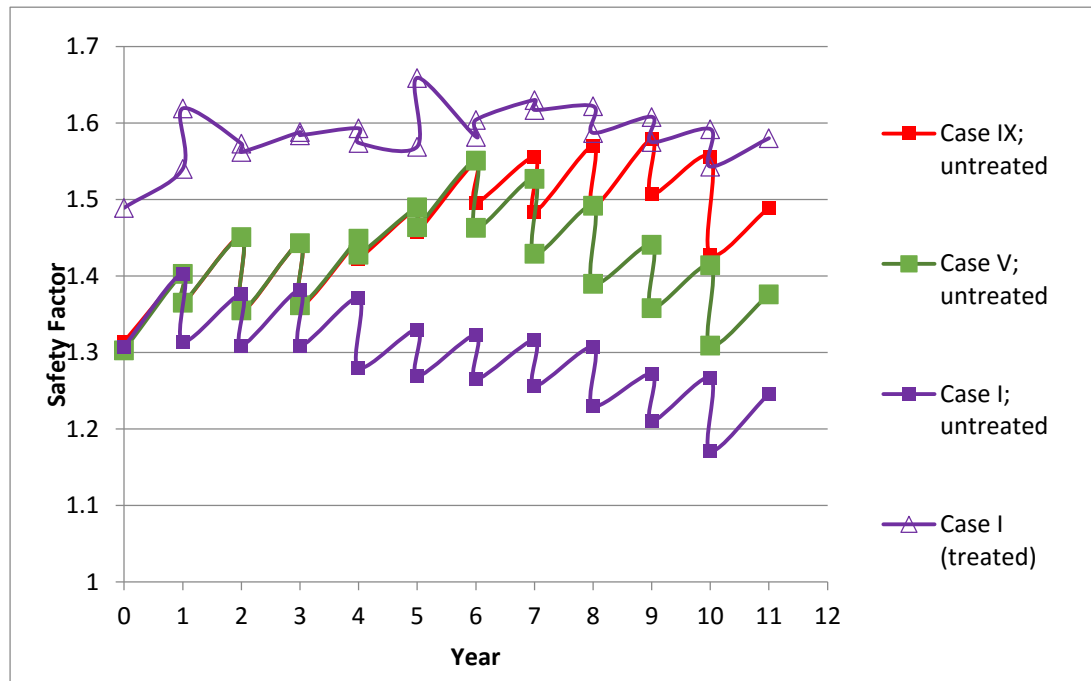


Figure 4-18 Variation of computed safety factor versus time for untreated and treated

7.5% (1CKD: 1B: 1OC).

Figures 4-19, 4-20 and 4-21 show the progressive movement of the slip surfaces as the construction proceeds. As the construction continues, the volume of tailings enclosed within the critical slip surface increases. Thus, the contribution of the tailings to the shearing resistance along the slip surface increases, and thereby the contribution of the treated tailings to the stability of the tailings dam increases.

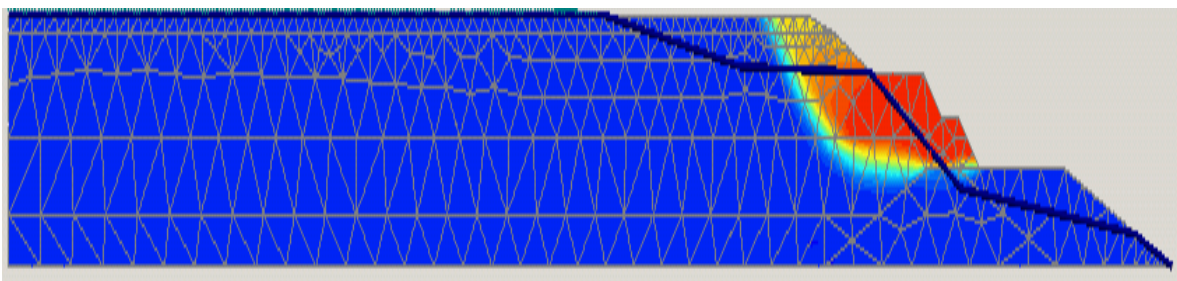




Figure 4-19 Slip surface location for first layer.

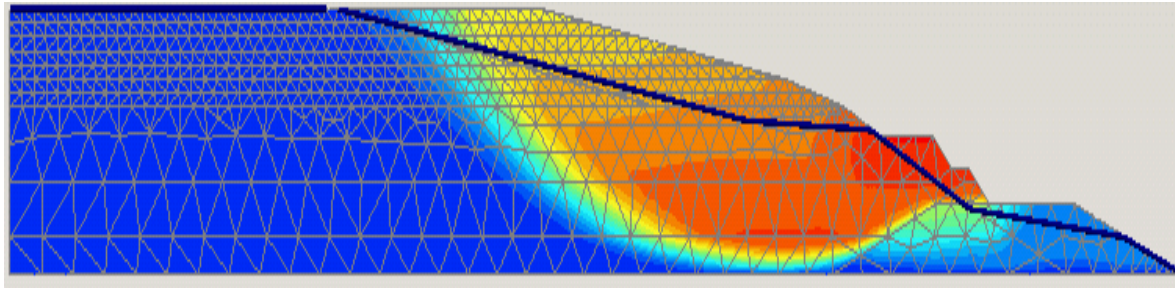


Figure 4-20 Slip surface location for 7<sup>th</sup> layer.

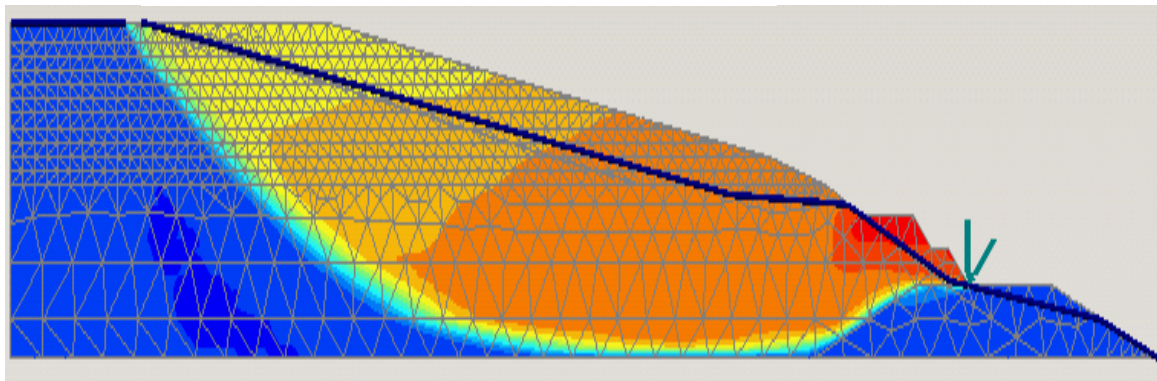


Figure 4-21 Slip surface location for 11<sup>th</sup> layer.

### 4.4.2.2 *Effect of Tailings Treatment on Settlement*

As was discussed in Chapter 3, the stiffness of the treated tailings increased substantially compared to the untreated tailings. This stiffness increase could have an important effect on the settlement of the tailings dams. In addition, the treatment reduces the hydraulic conductivity of the tailings, which could reduce the consolidation settlement. The effect of the tailings treatment on the settlement of the tailings dam is demonstrated in Figures 4-22 and 4-23. As can be noted from these figures, the effect of this treatment method on the settlement was profound. For example, the settlement after completion of consolidation of 11<sup>th</sup> layer for Case I of untreated soil was 3.60m (Figure 4-22), while the settlement of the same case but with treated tailings stiffness was only 2.16 m (Figure 4-23), i.e., dam settlement decreased by almost 40% due to the tailings treatment. The excellent performance of the tailings treatment scheme in terms of

increased SF and reduced settlement indicates the suitability of this method for the upstream construction of tailings dam. The implementation of this method can eliminate the need to use large volume of borrowed material for final countering before decommissioning. It will also minimize the potential for differential and creep settlements.

Indeed, the tailings treatment investigated in the current study proved successful in not only increasing the overall stability of Aitik tailings dam for the proposed staged construction plan, around 25% increase, meeting the conventional minimum safety factor of 1.5 but also in decreasing the amount of settlement by around 40%.

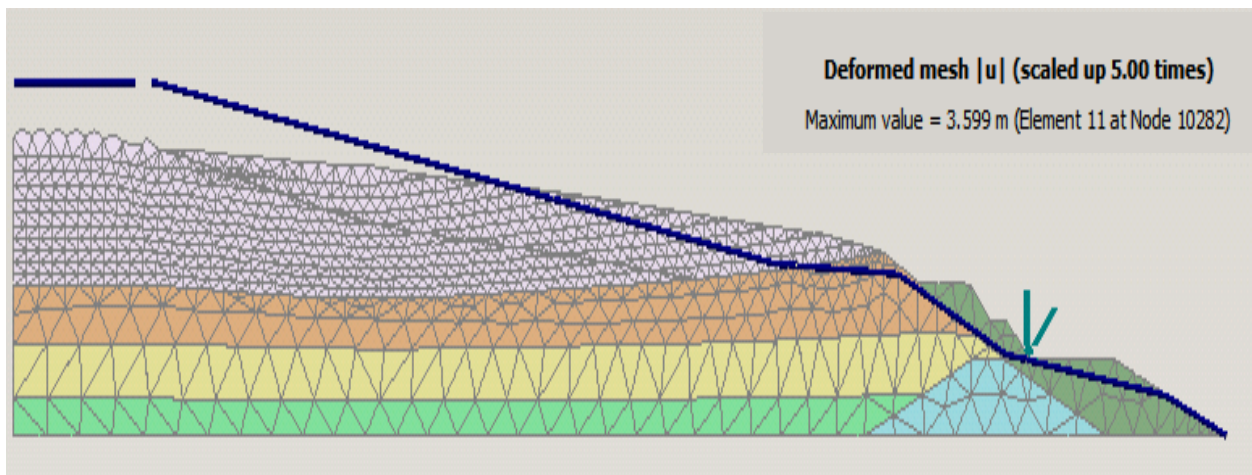


Figure 4-22 Total settlement for case I, untreated, after consolidation of 11<sup>th</sup> layer took place.

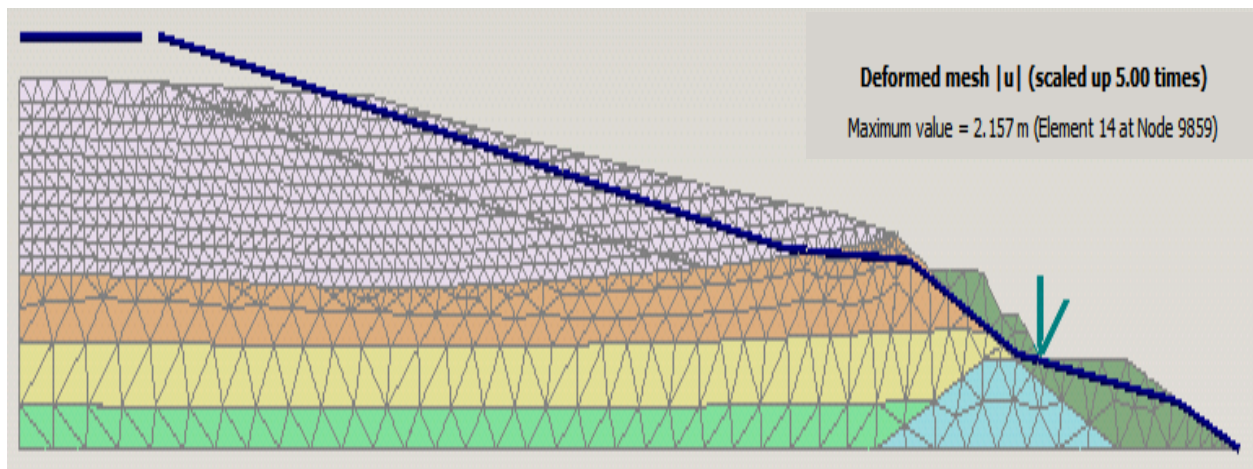


Figure 4-23 Total settlement for case I, treated, after time of consolidation of 11<sup>th</sup> layer had finished

### 4.4.2.3 Effects of Tailings Stiffness on Safety Factor

The mechanical properties of the tailings treated with the mix proportion (1CKD:1B:1OC) were used in the numerical model to investigate the effect of the elastic modulus of the treated tailings,  $E$  on the performance of the dam. The stiffness of the treated tailings was evaluated from the Oedometer test,  $E_{\text{oed}}$ , for the applied pressure range 100-400 kPa and was found to be 30.0 MPa. This value was used to represent the stiffness of the tailings in the numerical model of Case IX. Additionally, the analysis was repeated considering reduced stiffness of 3.0 MPa. Figure 4-24 presents the variation of the safety factor with time for the two cases. It is from Figure 4-24 that varying  $E_{\text{oed}}$  from 3.0 to 30.0 MPa had little to no observed effect on the safety factor. The effect of stiffer tailings was a small reduction in the EPP, which resulted in a small increase in SF.

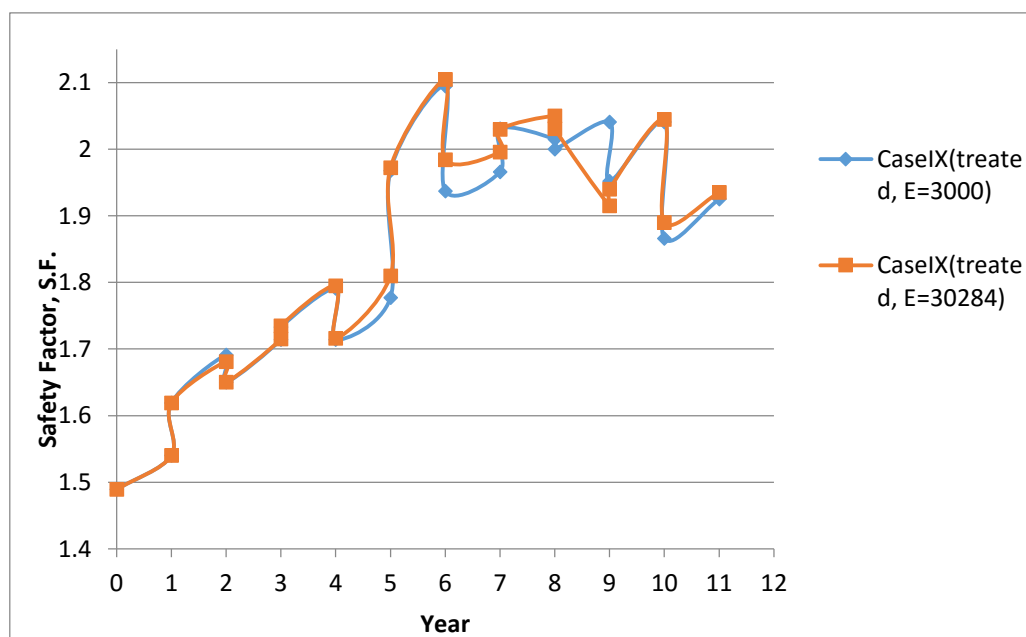




Figure 4-24 Safety factor of case IX with treated tailings shear strength and hydraulic conductivity parameters.

### 4.4.2.4 Effects of Changing Tailings $k$ Value

Figures 3-26 and 3-27 in Chapter 3 demonstrated that the hydraulic conductivity is affected by the value of void's ratio  $e$  and to the applied effective stress. As the tailings containment is raised, effective stresses increase and the void ratio decrease with depth. Therefore, the hydraulic conductivity decreases as well. This interaction between the three parameters is not accounted for in the Mohr-Coulomb model in Plaxis 2D package. Therefore, it is interesting to explore the effect of using a constant value of  $k$  across the depth throughout the analysis on the calculated response of the dam. For this purpose, Case I of the untreated soil was re-analyzed considering different values of  $k$ . In the first analysis,  $k = 7.866 \times 10^{-07}$  cm/sec, which corresponds to effective stress of 5kPa was used and in the second analysis  $k = 2.11 \times 10^{-08}$  cm/sec, which corresponds to an effective stress of 800kPa, was used. Figure 4-25 displays the variation of safety factors versus time for both cases. As expected, using a lower hydraulic conductivity yielded a lower safety factor.

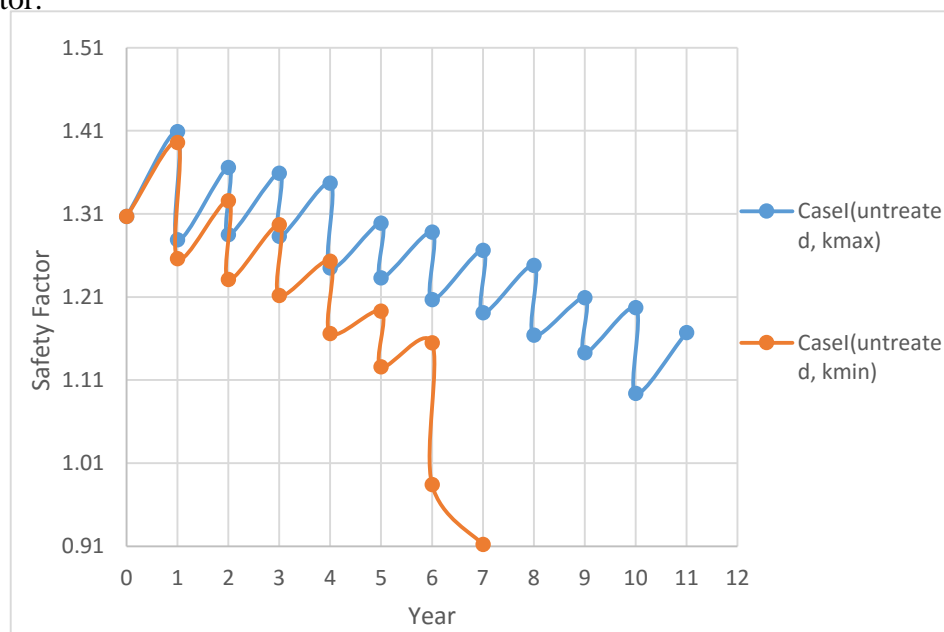


Figure 4-25 Case I S.F variation versus number of years considering upper and lower boundary of hydraulic conductivity.

Figure 4-26 shows the variation of excess pore pressure for the two cases at the end of 6<sup>th</sup> year. The results presented in Figure 4-26 demonstrate that the lower hydraulic conductivity resulted in higher excess pore pressure that is as much as twice the one of the upper k value, 140 and 76kPa, respectively. Moreover, when the low k value was used, the distribution of EPP changed and the contours of EPP expanded into the critical zone within which potential slip surfaces exist. This resulted in reduced effective stresses in this region and thus a reduced safety factor. Therefore, when high k value is used, slope stability analysis may yield too optimistic S.Fs that are non-conservative. In fact, when lowest k value was used in this study, failure is indicated to have initiated only six years from the construction beginning (Figure 4-25). This demonstrate how important the selection of this parameter is when slope stability of tailings dams is concerned. This is because as the hydraulic conductivity increase excess pore water dissipates quickly. Therefore, the value of k used has to be cautiously selected to model tailings dams' stability and should represent the range of stresses anticipated in the field. For conservative estimate of safety factor of the tailings dam, low value of k should be used in the analysis. However, it is much more appropriate and effective to simulate the change in k with effective stress which is something MC failure criterion in PLAXIS 2D is lacking.

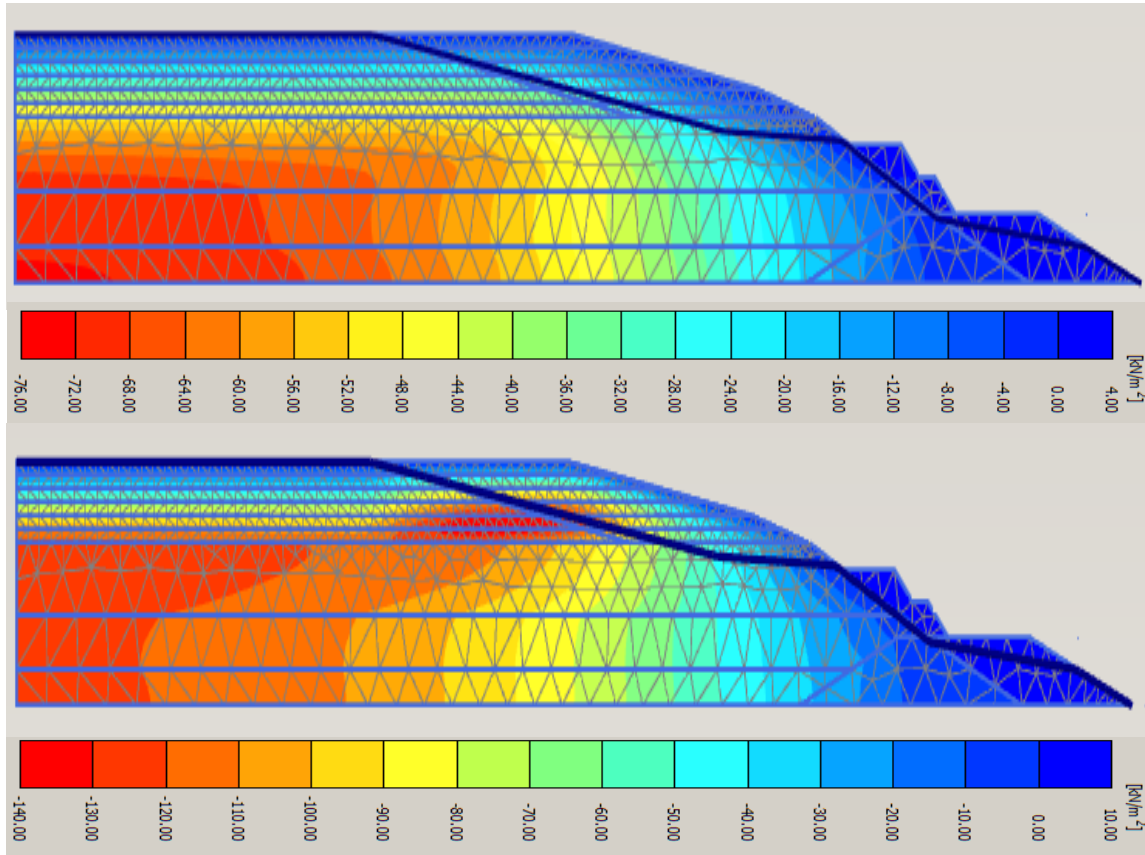


Figure 4-26 EPP profiles for k max, top, and k min, bottom of the consider case I for untreated tailings.

### 4.5 Summary and Conclusion

The effect of tailings treatment on the stability of tailings dams was investigated numerically. The outcomes of the numerical analyses work can be summarized as follows;

- Soil treatment scheme adopted turned out promising results with regard to slope stability enhancement. The S.F of the treated tailings was consistently between 20 and 25% higher than that of untreated tailings.
- The soil treatment proposed reduced the maximum settlement of the TSF by almost 40% by the end of the 11<sup>th</sup> year. This can, in turn, help reduce the contouring cost by having

reduced the settlement and therefore the amount of borrowed materials required for final leveling for closure. The reduced contouring cost will help pay off the cost spent on the soil treatment proposed. Also, soil treatment proposed will eliminate the need for other stabilizing measures such as berms, rock-fills, and compaction.

- Soil stiffness has small effect on the safety factor but has a profound impact on the settlement.
- Hydraulic conductivity value should be cautiously selected for conducting any slope stability problem as its value will dictate how much water seeps out due to staged construction typically followed or any hydraulic energy change in the soil. This aspect is most important when analyzing stability of tailings dams where  $k$  value can vary substantially from one point/time to another.

## **Chapter 5**

# **“Conclusions and Recommendations”**

### 5.1 Summary

This thesis is focused on investigating the stability of tailings dams and evaluating possible schemes for enhancing their stability. A comprehensive literature review was conducted to gather information on the different construction schemes of tailings dams and review the available information on the incidents and failure occasions.

The comprehensive literature review demonstrated that tailings dams are commonly built to a lower degree of engineering practice than conventional water dams. Tailings dams are commonly constructed employing the staged construction technique, which causes generation of EPP and subsequent dissipation in the saturated tailings. This is accompanied by changes in the effective stresses, which affects the stability of the tailings dam and its safety factor. It was also summarized that the tailings disposal methods employed in practice are: wet, dry and a combination of wet and dry disposal. Most frequently used method is the wet deposition due to its efficiency and cost effectiveness. Three construction methods are employed in the tailings disposal and construction of tailings dams: upstream, downstream and centerline. The most popular method is the upstream construction; however, tailings dams constructed using this technique experienced the highest recorded number of incidents and failures. Failure triggering mechanisms mostly involve pore water pressure generation, which may compromise tailings dams' stability. The literature review also showed that mine tailings are mostly with low PI, low hydraulic conductivity, and in loose or soft state. In addition, they are reported to exhibit shear strain softening, reduction in effective stresses and increase in pore water pressure when loaded. Given the observed behavior of tailings dams and properties of tailings, effective stress analysis appears to be the most suitable approach for evaluating stability of tailings dams. Several numerical investigations are reported in the literature, which employed the finite element method

to analyze the stability of tailings dams. The program PLAXIS 2D program, among other computer programs, is utilized in industry for the analysis and design of tailings dams.

Based on the findings of the literature review, it was concluded that the upstream construction technique is the most efficient technique for construction of tailings dams, but dams constructed according to this technique experienced the largest number of failure. Therefore, this thesis focuses on improving the stability of upstream tailings dams using soil additives.

The thesis work comprised two phases: a laboratory experimental phase and a numerical investigation phase. The experimental phase of this study involved laboratory tests that were conducted to characterize mine tailings and to investigate the changes in their properties upon stabilization with traditional and non-traditional additives: namely, emulsified polymer and a mixture composed of Cement Kiln Dust, CKD, and re-cycled Gypsum. The properties of both untreated and treated tailings material were established from Gradation tests, Oedometer tests, Direct Shear tests, and Unconfined Compression tests. The properties of treated and untreated tailings were used to establish finite element models employing the commercial code PLAXIS 2D to simulate the behavior of a tailings dam reported in a well-documented case study in order to evaluate the effectiveness of the proposed treatment scheme in enhancing the stability of tailings dams.

Traditional and non-traditional soil additives were used to enhance the geotechnical parameters of silty tailings. The non-traditional additive used was a commercial one in liquid state. The traditional additive was a mixture of recycled gypsum, B, and cement kiln dust, CKD.

Oedometer, UCS and DS tests were conducted. In addition, a numerical model was established for the analysis of a tailings dam described in a well-documented case study. The dam was constructed using the staged construction of 11 raises. Each raise was 3 meters thick constructed

in 10 days and allowed to consolidate for 355 days. The predictions of the model developed in this thesis were compared with that model by Ormann et al. (2013), and were found to be in excellent agreement. Afterwards, the properties established from the testing program were used to simulate the tailings behavior in the same numerical model to evaluate the improvement in the dam's stability and increase in its safety factor.

### 5.2 Conclusion

The following conclusions were drawn based on the experimental and numerical investigations.

- 1- Soil improvement by mixing the slurried tailings with cementitious additives, thereby increasing the dry density and adding cohesion to the tailings matrix, was examined. The tailings treated with emulsified polymer were more compressible than the tailings material and therefore its use was deemed inappropriate for this application.
- 2- The mechanical properties of tailings treated with a mixture of recycled gypsum, B, and cement kiln dust, CKD were found to be improved compared to original tailings. The unconfined compression tests showed that the tailings compressive strength increased as the mixture percentage increased. However, it decreased with any increase in the B ratio into the admixture, which was attributed to the weak bond of B due to its large surface area.
- 3- An amended admixture of 7.5% by total mass of the tailings with equal proportions of Ordinary Cement, CKD and B was considered for improving the tailings admixture. Based on Oedometer and DS tests, the stiffness of the treated tailings increased by more than 300%; however, this was accompanied by an order of magnitude reduction in  $k$  value.



- 4- The numerical model demonstrated that this admixture (OC, CKD and B) added at 7.5% by total weight to the tailings would result in an average increase of 25% in the overall SF of the dam and around 40% decrease in the total settlement. Furthermore, employing this treatment would eliminate the need to use rock-filling, reinforcing berms or flatter slope to attain a satisfactory safety factor.

The author believes that tailings improvement method proposed will have additional positive effects that were not considered within the scope of this study such as increasing the cyclic resistance of the tailings, reducing creep settlement, and reducing the differential settlement of the tailings impoundment.

### 5.3 Ideas for Future Research

Based on the results of this study, there are numbers of issues that need more investigation.

These are summarized below:

- 1- For tailings dams in cold regions, freeze and thaw can have tremendous effect on the consolidation and expansion behavior of the soil and thus the intergranular forces. This is believed to have side effects on the strength properties of the treated material.
- 2- Salinity and pH studies are lacking in this field. It was reported that the pH value can change the recorded angle of internal friction which means either under or overestimation of the stability of tailings dams.
- 3- Creep behavior is not well understood in the field of mine tailings dam as those dams are relatively new, manmade structures. In addition, sustainability is a major factor which may have negative effects on the settlement behavior of typically angular to sub-angular

tailings where concentration of stresses could lead to particle crushing and associated volume reduction.

- 4- Liquefaction of mine tailings dams can be assessed by employing field testing, lab testing and numerical modelling. Field testing that can be used are SPT and CPT with retrieving samples to the lab to determine fine contents and to determine the CRR as well as the exhibited strain behavior. Once done, the numerical model can be used with a constitutive model that suits the observed lab behavior to assess whether static liquefaction is initiated due to the staged construction and the associated EPP generation or not by defining the hydraulic boundary condition of the field. Cyclic mobility can also be investigated since the tailings are reported to show the strain softening behavior in conventional lab testing. An earthquake record can be used to measure the EPP within the numerical model and to see whether liquefaction happens or not. This is very important step to check before a layer is added especially in seismically active areas.
- 5- Effects of high stresses such as 2MPa or 3MPa, which are within the range of tailings dam's stresses, are lacking. Those very high stresses can have a downside effect on the shear strength parameters as well as the stiffness parameters.

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1888. doi:10.1002/nme.1406

## **Curriculum Vitae**

**Name:** YAZEED ABDULLAH M AL-SHAREDAH

**Date of birth:** 5/6/1989

**Major:** Civil Engineering

**Current Occupation:** ME.Sc. Student at Western Ontario University

### **Education:**

September 2013 – December 2015                      - M.E.Sc. in Civil Engineering

Western Ontario University, London-ON, Canada

Graduation year: 2015

September 2007 – July 2011                      - Bachelor in Civil Engineering

Qassim University, Qassim, Saudi Arabia

Graduation year: 2011

### **Experience:**

- Employed by Saudi Oger Company as site engineer: 7 months (cooperative training program).
- Site engineer at Yard Engineering consultant office for civil engineering in the construction of a water filtration station project located in Qassim-Saudi Arabia. My duties included approving the rebars work after ensuring they meet the project's drawings and the standards of Ministry of Water and Electricity.

- Teaching assistant at college of engineering in Qassim university (October 2011- December 2012).

- Graduate researcher at Western University (September 2013- December 2015).

### **Overview:**

I'm currently finishing my M.Sc. Studies at the University of Western Ontario. My thesis topic is "Slope stability enhancement for a mine tailings dam; Laboratory testing and Numerical modelling". Tailings dams are one of the subjects that is of problems. The tailings are mine by-products which are useless and require disposal. They are typically mixed with water and pumped into some containments to ease their disposal. This situation led to some thousands of dams being built utilizing staged construction and embodying the tailings as a construction material. Over time, unplanned or poorly planned disposal have caused some tailings dams instabilities where millions of the contained wet tailings were released to the surrounding environment causing death, environment pollution and economical loss (Zardari, 2013, Rico et Al, 2008). These reported incidents attracted myself and my supervisor to come up with a stabilizing mean for those tailings to promote safe construction and operation practice for tailings dams. I developed a comprehensive lab testing program to investigate the properties of slurried mine tailings and how they behave from the time they are deposited till the time they transform to soil. I was able to define the state of soil formation based on column sedimentation test. Afterwards, I tested the tailings sediment for their geotechnical properties to establish the data required for Mohr-Coulomb model. I then investigated the tailings improvement when traditional and non-traditional additives were added to the tailings matrix. I used emulsified polymers as non-traditional additives and CKD: B as the traditional one. The results indicate that the

traditional additives had a substantial improvement on the tailings shear strength, and therefore the shear stiffness, parameters. Finally, the treated and untreated tailings parameters were used in a validated numerical model to verify whether or not the stability of the tailings dam was improved or not. The used of CKD: B admixture showed that the slope stability of improved tailings was around 30% higher than that of untreated soil which shows how competitive and useful this technique could be should the stability of tailings dams be concerned.

### **Field of Interests:**

- 1- Foundation engineering.
- 2- Geotechnical earthquake engineering.
- 3- Site micro-zonation
- 4- Site investigation.
- 5- Laboratory testing.
- 6- Slope stability.

### **Summary of qualifications:**

- High developed computer skills.
- Strong aptitude for learning new computer technology.
- Proven ability to work in a team environment.
- Self-starter and able to work independently.

### **Skills:**

I am a cooperative person who can work in a group environment. I have strong communication skills that will benefit the team with which I work. I can work independently as well. I speak English fluently in addition to my mother tongue. I am an energetic person who finds ways to

overcome difficulties and obstacles which is essential character of a leader. Moreover, I have a strong grasp of the following computer programs:

- PLAXIS 2D (My ME.Sc. thesis includes numerical modelling of staged construction of a tailings dams)
- Structure program analysis (Sap2000)
- Geo-Studio2012 (I attended a workshop in Toronto and was exposed to a lot of hands-on examples on the various interactive components of the package in which I excelled my knowledge about the program). Besides, I used the program in a graduate course taught by Prof. M. H. El-Naggar for slope stability calculations.
- Response2000 (A program used to get the design structural parameters for any concrete section using the interaction diagrams)
- AutoCAD 2-D
- Deepsoil; a program that I used in a graduate course at Western taught by Prof. M. H. El-Naggar which is developed to get the site response to an earthquake. It is 1D program that is suitable for only horizontal ground surfaces.
- MS-Office 2013(Word, Excel and PowerPoint).

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Signature:

Yazeed Alsharedah

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Date:

22/10/2015

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